REPORT
ROYAL COMMISSION

QUEBEC BRIDGE
INQUIRY
1908

EGRArdagh

ROYAL COMMISSION

QUEBEC BRIDGE INQUIRY

REPORT

ALSO

REPORT ON DESIGN OF QUEBEC BRIDGE

BY

C. C. SCHNEIDER

PRINTED BY ORDER OF PARLIAMENT

OTTAWA
PRINTED BY S. E. DAWSON, PRINTER TO THE KING'S MOST
EXCELLENT MAJESTY
1908

[No. 154—Vol. I—1908]

TABLE OF CONTENTS

COPY OF COMMISSION.

CANADA.

GREY.

[L.S.]

EDWARD THE SEVENTH, *by the Grace of God, of the United Kingdom of Great Britain and Ireland, and of the British Dominions beyond the Seas, King, Defender of the Faith, Emperor of India.*

To all to whom these presents shall come or whom the same may in anywise concern,

GREETING:

Whereas, in and by an order of Our Governor General in Council, bearing date the thirty-first day of August, in the year of Our Lord one thousand nine hundred and seven, provision has been made for an investigation by Our Commissioners therein and hereinafter named into the cause of the collapse of the Quebec Bridge, in the course of construction over the St. Lawrence River, near the City of Quebec, in the Province of Quebec, on the 29th August, 1907, and into all matters incidental thereto.

Now know ye, that by and with the advice of Our Privy Council for Canada, We do by these presents nominate, constitute and appoint Henry Holgate, of the City of Montreal, in the Province of Quebec, Civil Engineer, John G. G. Kerry, of Campbellford, in the Province of Ontario, Civil Engineer, and John Galbraith, of the City of Toronto, in the Province of Ontario, Dean of the Faculty of Applied Science and Engineering and Professor of Engineering in the University of Toronto, to be Our Commissioners to conduct such inquiry.

To have, hold, exercise and enjoy the said office, place and trust unto the said Henry Holgate, John G. G. Kerry and John Galbraith, together with the rights, powers, privileges and emoluments unto the said office, place and trust, of right and by law appertaining, during pleasure.

And we do hereby, under the authority of the Enquiries Act, Chapter 104, of the Revised Statutes, 1906, confer upon Our said Commissioners the power of summoning before them any witnesses, and of requiring them to give evidence on oath, or on solemn affirmation, if they are persons entitled to affirm in civil matters, and orally or in writing, and to produce such documents and things as Our said Commissioners shall deem requisite to the full investigation of the matters into which they are hereby appointed to examine.

And We do hereby require and direct Our said Commissioners to report to Our Governor General in Council the result of their investigation, together with the evidence taken before them, and any opinion they may see fit to express thereon.

In testimony whereof, We have caused these Our letters to be made patent, and the Great Seal of Canada to be hereunto affixed. Witness, Our Right Trusty and Right Well-beloved Cousin the Right Honourable Sir Albert Henry George, Earl Grey, Viscount Howick, Baron Grey of Howick, in the County of Northumberland, in the Peerage of the United Kingdom, and a Baronet; Knight Grand Cross of Our Most Distinguished Order of Saint Michael and Saint George, &c., &c., Governor General and Commander in Chief of Our Dominion of Canada.

At Our Government House, in Our City of Ottawa, this thirty-first day of August, in the year of Our Lord One thousand nine hundred and seven, and in the Seventh year of Our Reign.

By Command.

F. COLSON,
Acting Under-Secretary of State.

EXTRACT *from a Report of the Committee of the Privy Council, approved by the Governor General on the 31st August, 1907.*

On a memorandum, dated 30th August, 1907, from the Acting Minister of Railways and Canals, representing that under date the 30th August, 1907, the Deputy Minister and Chief Engineer of the Department of Railways and Canals advises that the Quebec Bridge, so-called, in course of construction over the St. Lawrence River near the City of Quebec, by the Quebec Bridge and Railway Company, collapsed on the 29th August, 1907, causing loss of life and property.

That he states that it is his opinion that a commission should issue to three competent engineers empowering them to make an investigation, under oath, into the cause of the collapse of such bridge, and into all matters incidental thereto, and that this action should be taken immediately in view of the grave situation and the circumstances of the case. He further suggests the names of Mr. Henry Holgate, Civil Engineer, of Montreal, Mr. J. G. G. Kerry, Civil Engineer, of Campbellford, Ont., and Professor John Galbraith, of the University of Toronto, as Commissioners for this purpose, and advises that the remuneration paid to each Commissioner be at the rate of Fifty Dollars a day and all expenses in connection therewith.

The Minister, concurring in the view taken by the Deputy Minister and Chief Engineer, recommends that authority be given, in pursuance of the Act of the Revised Statutes of Canada, 1906, Chapter 104, Part 2, "An Act respecting public and departmental inquiries," to appoint Messrs. Holgate, Kerry and Galbraith as Commissioners to investigate and report upon the said matter, such investigation and report—but without thereby limiting the scope of the inquiry—to embrace and especially deal with the several questions suggested by the Chief Engineer.

The Minister further recommends that the salary to be paid to each of the said Commissioners be at the rate of Fifty Dollars ($50.00) a day for the days of actual service in connection with this inquiry, together with all reasonable living and travelling expenses defrayed in connection therewith.

The Committee submit the same for approval.

F. K. BENNETTS,
Ass't Clerk of the Privy Council.

REPORT TO HIS EXCELLENCY THE GOVERNOR GENERAL IN COUNCIL.

MAY IT PLEASE YOUR EXCELLENCY:

The Royal Commission appointed by commission dated the thirty-first day of August, A.D. 1907, to inquire into the cause of the collapse of the Quebec bridge, begs to present its report as follows:—

The members of the commission were appointed on August 30, 1907, the day following the accident, two of them proceeding to Quebec the same day, the third member arriving there on September 4. The formal commission was received on September 9. The taking of evidence at Quebec was commenced on the afternoon of September 9, and continued until September 24. On September 25 the commission went to Ottawa, and took evidence on September 26 and September 27. An adjournment was taken for the week ending October 5. On October 7 the commission reassembled in Quebec, and engaged in further examination of the wrecked structure and in study of the plans and documents. On October 14 the commission met in New York, and commenced the first examination of Mr. Theodore Cooper, consulting engineer of the Quebec Bridge Company, which continued until October 22. From October 23 until November 22 the commission was engaged in the taking of evidence and the collection of information in Phœnixville and Philadelphia. During this period two members of the commission visited the works of the Central Iron and Steel Company at Harrisburg, Pa., and other steel and bridge works which had no direct connection with the manufacture of the Quebec bridge were inspected. A second visit was paid to Quebec from November 28 to December 3, and on December 3 one member of the commission visited New York to further examine Mr. Cooper, returning December 6. On January 14 two members of the commission went to Phœnixville in order to make certain tests, returning on January 23. Since November 23, with the exceptions above mentioned, the time of the commissioners has been spent in Montreal in examination and discussion of evidence and in preparing this report.

We understand that the commission instructs us to determine to the best of our ability the cause of the collapse of the Quebec bridge, and to thoroughly investigate any matters appertaining thereto which might enable us to explain that cause. We do not think that either the general design of the Quebec bridge, the methods of financing the enterprise, the payments of money that have been made to or by the company or in its interest, or the obligations that the company has undertaken under various contracts and agreements have direct connection with the fall of the bridge. In the course of our investigations we have secured a large amount of general information on these and other matters not directly pertinent to the object of the inquiry, some of which have been introduced into this report so that the history of the undertaking might be more readily followed. We have not considered the scope of our inquiry limited concerning any matters which, in our judgment, related to the collapse of the bridge.

Some of our various inquiries have yielded negative results, but these are dealt with at some length in the report to make it clear that the subjects of these inquiries have not been overlooked.

In carrying out our instructions we have made the following investigations:—

(*a*) A study of the history of the Quebec Bridge and Railway Company, the evidence at our disposal being copies of the various public acts concerning it, the minutes of the directors' meetings, the reports of its officials, its annual reports, its correspondence and copies of the agreements and contracts that it has made.

(b) A perusal of the entire correspondence on file in the offices of the Quebec Bridge and Railway Company, the Phœnix Bridge Company and Mr. Theodore Cooper.

(c) A study of the working organizations of the Quebec Bridge and Railway Company, the Phœnix Bridge Company and the Phœnix Iron Company. This involved the hearing of a number of witnesses under oath, and the examination of the various documents produced by these witnesses on direction of the commission and filed as exhibits.

(d) A personal inspection of the furnaces and rolling mills by which most of the metal that was used in the bridge was produced. The testing equipment at each of the works was examined, and the file of the records of tests made by the inspectors during production was gone over.

(e) A study of the methods used in the fabrication, transportation and erection of the bridge. This consisted of inspection of the shops of the Phœnix Iron Company, in which all the metal was fabricated, and an examination of the plans, records, correspondence and photographs on file in the office of the Phœnix Bridge Company. The fabricated material for the north half of the bridge was also inspected, and check measurements were taken to determine certain questions of workmanship.

(f) A study of the errors in workmanship detected by the several inspectors during the progress of the work, the evidence available being the record books kept by the shop inspectors for the Phœnix Bridge Company and for the Quebec Bridge and Railway Company, the 'field corrections' sent by the Phœnix Bridge Company's resident engineer to the erection department of that company, and the weekly reports made by the inspector of erection for the Quebec Bridge and Railway Company to the consulting engineer.

(g) An inquiry into the history of the erection of the bridge. This inquiry was made by obtaining direct evidence from witnesses under oath and by tracing out through records and correspondence the details of all the major difficulties that had occurred in the course of construction.

(h) An endeavour to obtain from eye-witnesses of the disaster all details concerning it. Some twenty-five witnesses were examined for this purpose.

(i) An examination of the meteorological records for the day of the accident and for some time previous. The records of the Observatory at Quebec and those kept by the Phœnix Bridge Company's staff were available for this purpose.

(j) A personal examination of the fallen structures made at different times and occupying several days, together with such surveys, check measurements and photographs as were considered necessary.

(k) A study of the methods adopted in the design of the bridge. This study required an inspection of the drafting office of the Phœnix Bridge Company and an examination of the mass of preliminary and final designs on file there. The sworn statements of all the senior engineers formed an important part of the inquiry.

(l) A checking of the stress sheets prepared in the offices of the Phœnix Bridge Company, by comparison with the results obtained by Mr. C. C. Schneider, consulting engineer, who was employed subsequent to the disaster by the Department of Railways and Canals to report to it upon the design of the bridge.

(m) A comparison of the organization and specifications used for the Quebec bridge with those used for existing great cantilever bridges on this continent.

(n) A replotting of the records of tests made on full-sized compression members, and a comparison of the design for the principal compression chords of the Quebec bridge with similar designs for other great cantilevers. In this connection special tests were made both by the Phœnix Bridge Company and by the commission, the details of which are given.

(o) A study of the theory of compression members; standard books, transactions of technical societies and professional journals being consulted. The purpose of this

part of the inquiry was to determine how thoroughly the designers of the bridge availed themselves of the professional knowledge at their disposal.

Your commissioners desire to acknowledge the hearty co-operation throughout the inquiry of all officials of the companies directly concerned. Messrs. Cooper, Szlapka, Deans and Hoare especially have, in our judgment, made every effort in their power to assist us to establish the facts and have not attempted to spare themselves.

Some clearly contradictory statements are to be found in the evidence given in the early days of the inquiry by certain witnesses on whom the burden of the disaster fell. These statements may be attributed to the nervous tension under which the witnesses were labouring at the time.

Your commissioners find:

(*a*) The collapse of the Quebec bridge resulted from the failure of the lower chords in the anchor arm near the main pier. The failure of these chords was due to their defective design.

(*b*) The stresses that caused the failure were not due to abnormal weather conditions or accident, but were such as might be expected in the regular course of erection.

(*c*) The design of the chords that failed was made by Mr. P. L. Szlapka, the designing engineer of the Phœnix Bridge Company.

(*d*) This design was examined and officially approved by Mr. Theodore Cooper, consulting engineer of the Quebec Bridge and Railway Company.

(*e*) The failure cannot be attributed directly to any cause other than errors in judgment on the part of these two engineers.

(*f*) These errors of judgment cannot be attributed either to lack of common professional knowledge, to neglect of duty, or to a desire to economize. The ability of the two engineers was tried in one of the most difficult professional problems of the day and proved to be insufficient for the task.

(*g*) We do not consider that the specifications for the work were satisfactory or sufficient, the unit stresses in particular being higher than any established by past practice. The specifications were accepted without protest by all interested.

(*h*) A grave error was made in assuming the dead load for the calculations at too low a value and not afterwards revising this assumption. This error was of sufficient magnitude to have required the condemnation of the bridge, even if the details of the lower chords had been of sufficient strength, because, if the bridge had been completed as designed, the actual stresses would have been considerably greater than those permitted by the specifications. This erroneous assumption was made by Mr. Szlapka and accepted by Mr. Cooper, and tended to hasten the disaster.

(*i*) We do not believe that the fall of the bridge could have been prevented by any action that might have been taken after August 27, 1907. Any effort to brace or take down the structure would have been impracticable owing to the manifest risk of human life involved.

(*j*) The loss of life on August 29, 1907, might have been prevented by the exercise of better judgment on the part of those in responsible charge of the work for the Quebec Bridge and Railway Company and for the Phœnix Bridge Company.

(*k*) The failure on the part of the Quebec Bridge and Railway Company to appoint an experienced bridge engineer to the position of chief engineer was a mistake. This resulted in a loose and inefficient supervision of all parts of the work on the part of the Quebec Bridge and Railway Company.

(*l*) The work done by the Phœnix Bridge Company in making the detail drawings and in planning and carrying out the erection, and by the Phœnix Iron Company in fabricating the material was good, and the steel used was of good quality. The serious defects were fundamental errors in design.

(*m*) No one connected with the general designing fully appreciated the magnitude of the work nor the insufficiency of the data upon which they were depending.

The special experimental studies and investigations that were required to confirm the judgment of the designers were not made.

(*n*) The professional knowledge of the present day concerning the action of steel columns under load is not sufficient to enable engineers to economically design such structures as the Quebec bridge. A bridge of the adopted span that will unquestionably be safe can be built, but in the present state of professional knowledge a considerably larger amount of metal would have to be used than might be required if our knowledge were more exact.

(*o*) The professional record of Mr. Cooper was such that his selection for the authoritative position that he occupied was warranted, and the complete confidence that was placed in his judgment by the officials of the Dominion government, the Quebec Bridge and Railway Company and the Phœnix Bridge Company was deserved.

Owing to the necessity of having the evidence taken in the United States sworn to before a British consul, written questions were submitted to each witness examined in the United States, and written answers were returned after an interval of some days.

The commission is greatly indebted to the following gentlemen who have most courteously furnished information: Mr. Charles Macdonald, formerly chief engineer of the Union Bridge Company, contractors for the superstructure of the Memphis cantilever bridge; Mr. H. W. Hodge, of Messrs. Boller & Hodge, engineers of the Monongahela cantilever bridge; Mr. Ralph Modjeski, of Messrs. Noble & Modjeski, engineers of the Thebes cantilever bridge; Messrs. Ingersoll & Seaman, of the Department of Bridges of the City of New York, and Messrs. Reynders & Kunz, of the Pennsylvania Steel Company, respectively, engineers and contractors for the superstructure of the Blackwell's Island cantilever bridge.

We are also indebted for professional advice and assistance to Professor Mansfield Merriman, Professor W. C. Kernot, Professor W. H. Burr, Professor Edgar Marburg, Professor H. M. MacKay, Professor G. F. Swain, and Messrs. W. R. Webster, T. K. Thomson and E. W. Stern, consulting engineers.

The technical investigations have been by far the most arduous and difficult part of our inquiry, and it is questionable whether they could have been brought to any conclusion without the assistance that these men of expert training and experience have so freely given.

We have set forth the facts which have convinced us of the soundness of our findings in the accompanying appendices, each of which is an independent discussion dealing at length with some one phase of our inquiry. The subjects of these appendices are as follows:—

1. The evidence given before the commission of inquiry;
2. The exhibits filed with the commission of inquiry;
3. The history of the Quebec Bridge and Railway Company up to the end of the month of August, 1903;
4. The Phœnix Bridge Company;
5. The effect of financial limitations upon the design of the bridge and a discussion of the evidence relating to this;
6. The history of the development of the specifications and a discussion of the evidence relating to it;
7. A description of the organizations and staffs maintained by the different corporations interested in the erection of the bridge;
8. A history of the development of the plans and of the methods followed in the designing offices;
9. Material, shop work and inspection;
10. Transportation and erection;
11. A discussion of the difficulties that arose during erection and of the events at the time of the collapse of the structure;

12. A description of the fallen structure;

13. An examination of the various full-sized column tests that have been made in America, accompanied by diagrams showing the results of these tests;

14. A comparison of the stresses in the several members of the main trusses computed from the bridge as finally designed, with the stresses authorized by the specifications. This comparison was made by Mr. C. C. Schneider, consulting engineer, and is embodied in his report to the Department of Railways and Canals.

15. A description of the various experimental researches that have been made in connection with the building of the Quebec bridge and during this inquiry;

16. A discussion of the theory of built-up compression members;

17. A comparison of the design for certain chords of the Quebec bridge with those for similar members of other great cantilever bridges illustrated with outline drawings of the bridges and copies of the shop drawings of the chords;

18. A critical discussion of certain parts of the specifications;

19. Miscellaneous information.

All which is respectfully submitted.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY.
J. GALBRAITH.

MONTREAL, February 20, 1908.

(NOTE.—Appendices Nos. 1 and 2 will be found in another volume.)

APPENDIX No. 3.

THE HISTORY OF THE QUEBEC BRIDGE AND RAILWAY CO. UP TO THE MONTH OF AUGUST, 1903.

The bridging of the St. Lawrence river at or near the city of Quebec has been a subject of consideration for many years.

In 1852 Mr. Edward William Serrell, the engineer of the Lewiston and Queenston suspension bridge, at the request of the City Council of Quebec, examined the locality, and in a very complete report recommended a site for a bridge which is practically the same as that finally selected by the Quebec Bridge Company. At this site it was proposed to erect a suspension bridge for both railway and highway traffic.

From time to time other engineers investigated this project, and in 1884 Mr. A. L. Light, who had recently completed the construction of the Quebec, Montreal, Ottawa and Occidental Railway, submitted a plan to the Quebec Board of Trade, which was endorsed by Mr. James Brunlees, M. Inst. C.E.

None of these schemes, however, were seriously considered, there being no good commercial reason at that time to warrant the carrying out of so great a project.

HISTORY OF LEGISLATION.

A company to be known as the Quebec Bridge Company was incorporated in 1887—50-51 Vic. chap. 98—with a capital of one million dollars and with power to issue bonds; the provisional directors being Hon. J. G. Ross, Lt.-Col. Rhodes, R. R. Dobell, Hon. Thomas McGreevy, Lt.-Col. J. B. Forsyth, Gaspard Lemoine, Eugene Chinic, H. M. Price, Joseph Israel Tarte and Cyrille Duquet.

The company was given power to build and operate a railway bridge across the St. Lawrence river and to adapt it to the use of foot-passengers and vehicles. It might also construct lines of railway to connect the bridge with existing or future railways on each side of the river. Work of construction was to be commenced within three years, and to be completed within six years of the passing of the Act. The site and all plans required the approval of the Governor in Council, and all tolls to be charged by the company were subject to similar approval. This Act provided that should a change in ownership take place, the property should continue to be operated under the provisions contained in it and in the Railway Act.

The Quebec Bridge Company was unable to carry out the work required by the Act of 1887, and in 1891 an Act of Parliament was passed (54-55 Vic., chap. 107), which revived and re-enacted the Act of Incorporation, but amended it to the extent that the work should be commenced within three years and completed within six years from the date of the passing of the Act, in July, 1891.

Again, the company was unable to carry out the project, and in 1897 an Act was passed (60-61 Vic., chap. 69), reviving previous legislation and extending the date of completion of the work to June, 1902.

The company again applied to parliament for extension of time, and by an Act of 1900 (63-64 Vic., chap. 115) the time for completion was extended to June, 1905.

On October 9, 1900, an order in council was passed authorizing an agreement to be entered into between the government and the Quebec Bridge Company, which provided for the granting of a subsidy of one million dollars to the Quebec Bridge Company, one-third of which sum was to be applicable to the substructure and approaches, and two-thirds to the superstructure. In this agreement, the company

undertook to complete the bridge, all plans to be subject to the approval of the Governor in Council. The work having already been commenced, the agreement provided that it should be completed by January 1, 1903, failure in complying with this condition to be followed by the forfeiture of all right or title to any part of the subsidy. Certain specifications which are signed by E. A. Hoare, M. Inst., C.E., chief engineer of the Quebec Bridge Company, and dated September 1, 1898, were made part of the agreement which was completed on November 12, 1900 (Subsidy Agreement 13988 Ex. 12).

The province of Quebec in March, 1900 (63 Vic., chap 2) granted a subsidy to the Quebec Bridge Company to the amount of $250,000, upon condition that the city of Quebec would grant a like amount; and on June 1, 1900, the city of Quebec voted a subsidy of $300,000 to the same company, provided that the company lay its terminus within the limits of the city of Quebec.

By Act of Parliament in 1903 (3 Edward VII., chap. 177), the name of the company was changed to the Quebec Bridge and Railway Company, and the work was declared to be for the general advantage of Canada. Further powers were granted, authority was given to issue preference shares, and the bond issue was fixed at $6,000,000, with the right to issue further bonds covering any property that might be thereafter acquired.

The company was also empowered to enter into agreement with the government of Canada in reference to a guarantee of the bonds of the company, and for granting and conveying the bridge and property of the company to the government. The time for completion was extended to July, 1910.

Pursuant to the power granted under the Act of 1903, the Quebec Bridge and Railway Company entered into an agreement with the government of Canada on October 19, 1903, which agreement was confirmed by Act of Parliament on October 24, 1903 (3 Edward VII., chap. 54). By this Act the government undertook to guarantee the bonds of the company, the bond issue was fixed at $6,678,000, and the company was authorized to redeem the outstanding stock on certain conditions. The number of directors was increased to eleven, and the Governor in Council had the right to appoint three of these. Nothing in this Act authorized the government, without consent of parliament previously obtained, to exercise its right to take over the undertaking.

The above is a brief summary of the legislation that has affected the company from its inception to this date (February 20, 1908).

HISTORY OF PROGRESS.

At the annual general meeting of the Quebec Bridge and Railway Company, held April 20, 1897, the president, Lt.-Col. J. B. Forsyth, reported that subsequently to 1888 Mr. E. A. Hoare had carefully surveyed the St. Lawrence river on both sides from Quebec to the vicinity of the Chaudière, and had reported that a bridge could be built at three sites, viz.:—

1st, at Cape Diamond;
2nd, at Point-a-Pizeau; and
3rd, near the mouth of the Chaudière river.

After consideration of Mr. Hoare's report by the board, the matter was referred to Mr. Walter Shanly, who visited the different sites, and reported in 1889 in favour of the third of those above mentioned. Mr. Collingwood Schreiber, the chief engineer of the Department of Railways and Canals, also endorsed the Chaudière site in his report of February 28, 1891, which report was presented to parliament (Return No. 16, Session of 1891). At this meeting the Chaudière site was finally adopted by the company. The president, Lt.-Col. Forsyth, having resigned, his place was taken by the Hon. S. N. Parent.

On June 16, 1897, Mr. E. A. Hoare, the engineer of the Quebec Bridge Company, wrote to the president of the Phœnix Bridge Company asking if any of their engineers expected to attend the annual convention of the American Society of Civil Engineers, which was to convene at Quebec on June 30; and if so, he asked that they call upon him to discuss a project for building a bridge over the St. Lawrence river near Quebec. Mr. John Sterling Deans, the chief engineer of the Phœnix Bridge Company, went to Quebec and met Mr. Hoare and others connected with the Quebec Bridge Company. Hon. R. R. Dobell, one of the directors of the company, took many of the visiting engineers on an excursion to the site and explained the project to them. Mr. Theodore Cooper was one of the party who visited Quebec at this time and then first learned of the proposed work, and on July 7, 1897, Mr. Deans, of the Phœnix Bridge Company, wrote to Mr. Hoare stating that Mr. Cooper would be glad to give the Quebec Bridge Company the benefit of his extended experience. As stated by Mr. Deans, Mr. Hoare promised to send him a profile of the river crossing at the proposed site, and other general information necessary for the purpose of preparing a tender on the work should his company be asked to make one. This Mr. Hoare did, and the matter was at once taken up by the Phœnix Bridge Company, and on November 30, 1897, they completed their first preliminary general plan for the bridge. This plan was altered, and on December 7, 1897, a new plan was completed, and was sent to Mr. Hoare.

The Quebec Bridge Company, early in 1898, applied to the Railway Committee of the Privy Council for approval of the plans and proposed site of the bridge, which application was filed in the department as No. 7349. The plan that accompanied this application is dated January 13, 1898, and is signed by Messrs. S. N. Parent, Ulric Barthe and E. A. Hoare, and as to the superstructure it is identical with the plan made by the Phœnix Bridge Company, and dated December 7, 1897.

The site of the bridge and the positions of the piers and abutments were approved as shown on the plans. The bridge had a clear width of span over the channel of 1,200 feet, and a clear height of 150 feet from extreme high water, the clear span between pier centres being 1,600 feet. The plans of all details were made subject to the approval of the chief engineer of the Department of Railways and Canals before work could be commenced, and also subject to the approval of the Governor in Council upon the joint report of the Minister of Railways and Canals and the Minister of Public Works. The order in council conveying this approval was signed May 16, 1898 (Ex. 2).

On July 2, 1898, the board of the Quebec Bridge Company passed a resolution instructing Mr. E. A. Hoare, their chief engineer, to put himself in communication with Mr. Schreiber, and the secretary was instructed to write to the Right Honourable Sir Wilfrid Laurier, asking him to give instructions to the chief engineer of the Department of Railways and Canals to put his bridge engineer in communication with Mr. Hoare, so that suitable specifications for the proposed bridge might be prepared, to be used when calling for tenders (Ex. 4). These instructions were carried out, and Mr. Hoare conferred with Mr. R. C. Douglas, the bridge engineer of the department, and the specifications were prepared. On August 26, 1898, these general specifications were submitted to Mr. Schreiber and were approved by him as quite satisfactory on August 31, 1898 (Ex. 5).

The specifications thus approved by the Department of Railways and Canals were printed by the Quebec Bridge Company under date of September 1, 1898, and are practically the same as those attached to the subsidy agreement of November 12, 1900; they include specifications for both the substructure and the superstructure.

On September 6, 1898, the Quebec Bridge Company instructed their secretary to issue circulars inviting tenders; the date for receiving the same was made January 1, 1899, but subsequently this was changed to March 1, 1899.

In accordance with these instructions, the secretary issued a circular (Ex. 6) sending with each copy a section of the river showing the clearances required, and also specifications for a cantilever bridge; if any tenderers proposed a suspension

bridge they were to furnish complete specifications. A form of tender was sent to each party, which called for lump sum prices both for substructure and superstructure.

In response to this circular, tenders were received from the Keystone Bridge Company, of Pittsburg, for a cantilever bridge; from the Dominion Bridge Company, of Montreal, for both a cantilever and a suspension bridge; from the Phœnix Bridge Company, of Phœnixville, for both a cantilever and a suspension bridge; from the Union Bridge Company, of New York, for a suspension bridge, and from the New Jersey Steel Company, of Trenton, for a cantilever bridge. Tenders for substructure were received from Wm. Davis & Sons, of Cardinal, Ont., and from the Engineering Contract Company, of New York. The New Jersey Steel Company subsequently withdrew their tender.

At this date, March, 1899, the Quebec Bridge Company were not in a position financially to let a contract for any portion of the proposed structure, but the board considered that the prospects of obtaining funds were sufficiently promising to warrant the calling for tenders.

The construction of this bridge, being a task of unpredecented magnitude, the board, on February 23, discussed the appointment of a consulting engineer, and the names of six prominent engineers were considered, with the result that the secretary was instructed to write to Theodore Cooper and to ask him if he would consent to act. This instruction was carried out on the same day.

On March 23, 1899, Hon. S. N. Parent, Mr. Hoare and Mr. Barthe met Mr. Cooper in New York, and it was arranged that Mr. Cooper would examine and report upon the plans and tenders received for a certain fee. This agreement was confirmed by interchange of letters.

All plans and tenders were accordingly sent to Mr. Cooper.

During the period when these plans and tenders were in the hands of Mr. Cooper, the Phœnix Bridge Company kept in close touch with Mr. Cooper and Mr. Hoare, and reference may be made to Mr. Deans' letters of April 14 and April 19, 1899, addressed to Mr. Hoare.

The correspondence of the officials of the Phœnix Bridge Company at this stage indicates a strong desire to obtain a favourable report from Mr. Cooper as a preliminary to securing the contract for the work at a later date, and the letters from the officials of the Quebec Bridge Company to the Phœnix Bridge Company indicate a desire to assist it in this direction.

The apparent reason for this state of affairs is that the Phœnix Bridge Company were, as far as we can learn, the only tenderers who felt and expressed confidence in the Quebec bridge project, and had prepared all of the preliminary plans for it. The Quebec Bridge Company therefore inclined more favourably towards them, and the relations were mutually friendly.

As to either party influencing Mr. Cooper or causing him to modify his ideas so as to favour any tender, such a suggestion is, in our opinion, quite out of the question, and we believe that Mr. Cooper made his decisions and gave his opinions with absolute honesty.

On June 23, 1899, Mr. Cooper reported to the Quebec Bridge Company upon the tender submitted (Exhibit 9), the following being an extract from his report:—

'From the facts and consideration as stated above, I find the cantilever superstructure plan of the Phœnix Bridge Company an exceedingly creditable plan from the point of view of its general proportions, outlines and its constructive features.

'I also find that it is designed in accordance with your specifications.

'The tender accompanying this plan is the lowest in price, and is the most favourable as to the prospective duties upon the materials to be used in its construction.

'I therefore hereby conclude and report that the cantilever superstructure plan of the Phœnix Bridge Company is the "best and cheapest" plan and proposal submitted to me for examination and report.

'I likewise report that the general plan and proposals for the substructure made by the Engineering Contract Company and by Messrs. Davis & Sons are both satisfactory and at favourable terms.'

Mr. Cooper also advised that further investigation be made by boring and by sinking trial shafts to determine the best position for the piers, and suggested that, as the surveys that had been made up to date were not sufficient, in any contract that might be made, there should be provision for changing the length of spans within reasonable limits, for modifying the carrying capacity of the structure and for increasing or decreasing the construction quantities.

Mr. Cooper's report of June 23, 1899, was received, and was laid before the board of the Quebec Bridge Company on June 29, when it was resolved:—

'That a copy of Mr. Cooper's report, with superstructure plan of the Phœnix Bridge Company, and the Keystone, and Wm. Davis & Sons' substructure plan be sent immediately to the Right Honourable Sir Wilfrid Laurier.'

No positive action was taken as to the tenders, and no one of these was formally accepted then or at a later date.

The full report of Mr. Cooper is appended (Exhibit 9), and it will be observed that he believed that the cantilever designs were the most favourable owing to their lower cost, and these designs were therefore more critically examined than were those of suspension bridges. The comparison of tenders was narrowed down by a process of elimination, to two cantilever designs, those of the Keystone Bridge Company and of the Phœnix Bridge Company, both of which were 'acceptable designs.' After making due allowance for cost of foundations so as to put the cost of superstructure on an even basis, Mr. Cooper found that the tender of the Keystone Bridge Company for superstructure was $2,462,119, and that of the Phœnix Bridge Company was $2,438,612, making a difference in favour of the Phœnix Bridge Company's tender of $23,507; if duty were charged, this amount would be further increased by $97,768, owing to the greater weight of steel in the Keystone design.

The estimated weight of steel as per tenders was:—

Keystone Bridge Company, in gross tons..	27,400
Phœnix Bridge Company, in gross tons..	22,956
Difference in favour of latter, gross tons..	4,444

The tenders show the average price of steel per gross ton as follows, all erected and complete:—

Phœnix Bridge Company..	$103 94
Keystone Bridge Company..	90 00

The tenders were lump sum prices for a completed structure, provided that the work was executed in accordance with the plans submitted and the unit prices of steel per ton were given in the tenders solely as a basis for computing progress estimates.

In view, however, of the fact that at a subsequent date a contract was made with the Phœnix Bridge Company at a price per pound and not on a lump sum basis, it should be noted that, having the above figures before them, the Quebec Bridge Company did not ask for new tenders for the steel work on a pound or ton basis, and also that the weight of the structure designed for the longer span overran the originally estimated weight by nearly 45 per cent.

Negotiations were commenced with the Phœnix Bridge Company, but that company would not enter into a contract on account of the financial conditions of the Quebec Bridge Company.

Mr. Deans expressed himself as having full confidence in the scheme as a business undertaking, and made efforts to assist the Quebec Bridge Company by endeavouring to interest prominent American bankers in the project; he was unsuccessful, and all the financial firms declined to invest in the securities of the Quebec Bridge Company

owing to the fact that the probable immediate returns would not warrant them in taking the matter up.

At this time, June, 1899, the Quebec Bridge Company had only a stock subscription of $50,352.69, of which $26,684.74 had already been expended for surveys and other expenses.

In his report of June 23, 1899, Mr. Cooper advised that more information be obtained with regard to the river-bed, so that the cost both of foundations and of superstructure might be closely estimated before the length of the main span was finally settled, and we direct your attention to all the evidence on this point, which clearly shows that at the time of calling for tenders there was not sufficient information to justify the action of the Quebec Bridge Company in fixing the positions of the main piers. On Mr. Cooper's advice further borings and examinations were made under the supervision of Mr. Hoare. Dr. Ami, of the Dominion Geological Survey made a report on these borings, which is appended.

The information thus obtained was transmitted to Mr. Cooper on January 14, and after studying it, he reported to Hon. S. N. Parent, on May 1, 1900 (Ex. 11) recommending a change of the main span from 1,600 feet to 1,800 feet, for the following reasons:—

'First: The construction of the larger and deeper piers of the 1,600 ft. span will require at least one more year than those for the 1,800 foot span.

'Second: The contingencies of the construction of the deeper piers in the deeper waters, where they might possibly be subject in their incomplete condition to the heavy ice floes of the main channel, would be far greater than for the piers further in shore.

'Third: The effect upon any future financing, by reducing the time of construction and minimizing the real and imaginary contingencies.'

Mr. Cooper estimated that the additional cost of the changes he advised would be $200,000 provided that modifications were made in the specifications, which, in his opinion, were both desirable and justifiable, and would in no manner reduce the carrying capacity of the structure or render it incapable of fully performing all its duties satisfactorily. (Ex. 11.)

Previously to the receipt of Mr. Cooper's second report, the board, on August 14, 1899, requested a meeting with the Phœnix Bridge Company's representative, and, on August 21, Mr. Deans met the board and discussed the situation then existing. On the following day the board decided to divide the work between the Phœnix Bridge Company and Mr. M. P. Davis. On August 23, the Hon. S. N. Parent wrote Mr. Deans stating that the Quebec Bridge Company was ready to enter into a contract with the Phœnix Bridge Company, upon certain conditions, which included the modification of the specifications, and the terms of payment. The Phœnix Bridge Company were to accept their share of the $1,500,000 of subsidies or their equivalent and the difference in bonds. Under the same date Mr. Deans wrote to the Hon. S. N. Parent extending the privilege of ordering the work in whole or in part at the unit prices named in the tender of March 1, 1899, for 'say one or two years,' on the understanding that the prices would be modified in accordance with the variations in the base price of metal and would be fixed by agreement between the engineers of the two companies at the date of the final order for each part of the bridge. In so far as the Phœnix Bridge Company was concerned, nothing came of these negotiations, but an agreement for the construction of the substructure was made at a later date with Mr. M. P. Davis somewhat on these lines.

Matters made no further progress until the following spring when, at a meeting of the board on April 5, 1900, Hon. Mr. Parent stated that before concluding the contract for the masonry there were questions to be settled with 'the prospective superstructure contractor, the Phœnix Bridge Company.' Messrs. Audette, Breaky and Lemoine were then delegated to meet Mr. M. P. Davis about his contract and

conditions of payment, and Messrs. Parent, Audette and Price were selected to represent the company at a meeting with the Phœnix Bridge Company, which was subsequently held at Mr. Cooper's office in New York.

The arrangements with Mr. M. P. Davis were concluded in the month of April, although the contract itself was not executed until June 19, 1900, and at the meeting in New York, just mentioned, which was held on April 12, 1900, an agreement was made and signed by the Hon. S. N. Parent, president of the Quebec Bridge Company, and Mr. John Sterling Deans, chief engineer of the Phœnix Bridge Company, whereby the Quebec Bridge Company awarded the contract for the construction of the superstructure and steel anchorages of the bridge to the Phœnix Bridge Company upon the cash prices tendered on March 1, 1899, subject to the modifications suggested by Mr. Deans in his letter to the Hon. S. N. Parent under date of August 23, 1899, the superstructure and steel anchorages to be ordered within three years from date. The Phœnix Bridge Company agreed to deliver the steel work for the anchorages within four months after the approval of detailed plans, the price to be fixed at the date of ordering the metal. This was done on June 15, and the price, which was 4·516 cents per pound, was fixed in accordance with the terms of Mr. Deans' letter of August 23, 1899, by a board consisting of Messrs. Deans, Cooper and Hoare.

The Phœnix Bridge Company also agreed to complete all general and detail plans for the entire superstructure with all possible speed.

This agreement was approved by the Quebec Company's board on April 21, 1900.

It appears, therefore, that the contract was awarded for the superstructure before Mr. Cooper had reported upon the necessary change in span, and that the agreement of April 12 was really not in accordance with the tender of March 1, 1899, in that this tender contemplated a lump sum price for the whole work, and not a price per pound; the details of this matter will be referred to further on.

Mr. Cooper's report (Exhibit 11) of May 1, 1900, was submitted to the board on May 5, and was adopted. At the same meeting they appointed Mr. Theodore Cooper consulting engineer to the company in accordance with terms and conditions contained in the minutes of the board of March 23, 1899. These terms and conditions, however, we note, only applied to examining and reporting upon certain plans submitted to Mr. Cooper, and the appointment then made was for a specific purpose and was not in the nature of a permanent appointment as consulting engineer.

Mr. Cooper objected at a later date to the arrangement of the terms of remuneration, and wrote to Mr. Hoare on July 26, 1901, suggesting as a basis of adjustment, that his services as consulting engineer from April 11, 1900, to the completion of the metal superstructure, be placed at a lump sum of $22,500, with an additional retaining fee of $2,500 for each year exceeding three years that his services were required, yearly payments to be not less than $3,750. This letter was submitted to the board, and on August 7, 1901, was approved. The actual payments made to Mr. Cooper are given in Exhibit 114.

At the board meeting of May 5, 1900, the following resolution was passed:—

'That the report of Theodore Cooper, consulting engineer, in date of May 1 instant, recommending an 1,800 foot span instead of 1,600 feet, be adopted, and that the Quebec Bridge Company's engineers give instructions to the Phœnix Bridge Company, contractors for the superstructure, to prepare plans accordingly without delay, and also that the contractors for substructure, William Davis & Sons, be informed of such modifications, and that the contract for substructure work will be modified accordingly.'

The Phœnix Bridge Company, by letters of May 9 and 16, 1900, accepted the modifications in the plans of the bridge advised by Mr. Cooper.

The memorandum already referred to, concerning prices (Ex. 14), dated New York, June 15, and signed by Messrs. Cooper, Hoare and Deans, was ratified by the board on July 5, 1900, and the president advised the appointment of an inspector at the rolling mills and machine shop, which was authorized.

On December 19, 1900, a second contract was entered into between the Quebec Bridge Company and the Phœnix Bridge Company covering the erection of the approach spans on each side of the river, the unit price being at 4·114 cents per pound erected and painted complete. (Exhibits 13 and 14.) On January 17, 1901, the board approved the above agreement. The report of the directors presented at the annual meeting of the company held on September 4, 1900, fully sets out what had been done up to that time (Ex. 19).

On October 2, 1900, the 'corner stone' of the Quebec bridge was laid, and the report of the directors at the annual meeting held September 3, 1901, is interesting in that it contains reports of progress on the substructure from Messrs. Cooper and Hoare. Mr. Cooper approves the progress of the work and adds that 'During the past year special studies have also been made of the main span, to improve and better the same in advance of the preparation of the final plans.' At that time the north anchor pier was about complete, the ground was being prepared for the north abutment and the north main pier was well under way.

Good progress on the work under contract, viz.: the substructure, the anchorages and the two approach spans was made during the following year, and at the annual meeting of the company held September 2, 1902, Mr. Hoare reported that the substructure on north shore was completed, that the abutment on south shore would be finished in a month, and that the south anchor pier was all finished except two courses of masonry. He also reported that the main pier on south shore was in progress, and that it had been found that a greater depth had to be reached to get a satisfactory foundation than was at first expected, and that in consequence it would take some time to complete this pier. The north approach span was in course of erection, and the material for the south approach span had been delivered.

On October 13, 1902, Mr. Cooper reported on the south main pier, and on February 3, 1903, he again reported, stating that the experience of the last two summers amply justified the change in the length of the main span from 1,600 to 1,800 feet.

Negotiations for the construction of the main span which, in the meantime, had not proceeded actively were now resumed with the Phœnix Bridge Company, and Mr. Deans wired the Hon. S. N. Parent, on May 11, 1903, that he would be in Quebec on the 15th and could go to Ottawa on the next day or on any other convenient day, as had been requested.

This visit to Ottawa was made on account of legislation proposed to be submitted to parliament in relation to the Quebec Bridge Company and the financial support to be given to it by the government; and the Phœnix Bridge Company desired to have the enactment of this legislation assured, before entering into any further contract with the Quebec Bridge Company.

The prospects for favourable legislation being satisfactory, articles of agreement were prepared and signed by the Quebec Bridge Company and by the Phœnix Bridge Company, on June 19, 1903 (Ex. 16), and were approved by the board of directors of the Quebec Bridge Company on the same day.

In transmitting the executed agreement, Mr. David Reeves, the president of the Phœnix Bridge Company, attached a letter of same date in which he states that the agreement is executed by his company upon the understanding that it shall not become operative until the legislation proposed shall have been enacted and financial arrangements insuring payments of estimates shall have been made to the satisfaction of his company. He agreed to go on with strain sheets and drawings as soon as the revised specifications with the formal approval of the government engineers were furnished to his company. These conditions were accepted by the Quebec Bridge Company.

In his suplementary report of June 23, 1899, Mr. Cooper advises :—

'It might also be desirable to ask the successful competitor to state what reductions, if any, could be made in the tender by certain modifications of the specifications.'

This indicates that changes in the Quebec Bridge Company's specifications were in Mr. Cooper's mind at that early date, and also that he considered the tender as a lump sum tender, and not otherwise.

On May 1, 1900 (subsequent to the awarding of the contract), Mr. Cooper suggested to Mr. Parent, in a letter, that he 'be instructed to make such modifications in the accepted competitive plan when adapted to the new lengths, as may tend to reduce the cost without reducing the carrying capacity or the stability of the structure.'

On June 2, 1903, Mr. Cooper transmitted certain amendments to the specifications attached to the subsidy contract of November 12, 1900, and gave his reasons for the proposed changes; as under section 2 of this agreement, any amendments of plans and specifications had to be approved by the Governor in Council, these amendments were submitted to Mr. Schreiber for examination. Mr. Schreiber, the chief engineer of the Department of Railways and Canals, examined the amended specifications, and communicated with the Minister of Railways and Canals on July 9, 1903. The Minister reported to council on July 16, 1903, and on July 21 an order in council was passed, embodying Mr. Schreiber's recommendations (Ex. 17). In his report Mr. Schreiber refers to discussions between himself and Mr. Cooper, the consulting engineer of the Quebec Bridge Company, involving certain modifications of the specification attached to the subsidy contract; he expresses his high regard for Mr. Cooper's professional standing, that gentleman being a man of repute and reliability. He adds: 'His modifications may, therefore, reasonably be considered to be in the best interests of the work.' Mr. Schreiber suggests that 'the department be authorized to employ a competent bridge engineer to examine from time to time the detailed drawings of each part of the bridge as prepared, and to approve of or correct them as to him may seem necessary, submitting them for final acceptance to the chief engineer of the Department of Railways and Canals.'

When a copy of the above order in council reached Mr. Cooper, he strenuously objected to the appointment of an engineer as suggested by Mr. Schreiber, saying: 'This puts me in the position of a subordinate, which I cannot accept.' Mr. Cooper, at the same time wrote to Mr. Schreiber: 'I do not see how such an engineer could facilitate the progress of the work or allow me to take any responsible steps independently of his consent.' Mr. Cooper then went to Ottawa to see Mr. Schreiber, and discussed the situation with him. In consequence Mr. Schreiber made a further recommendation, and an order in council was passed August 15, 1903 (Ex. 18) which directed that, provided the efficiency of the structure be fully maintained up to that defined in the original specifications attached to the company's contract (Ex. 12), the new loadings proposed by the Quebec Bridge Company's consulting engineer be accepted, &c.; and that all plans be submitted to the chief engineer, and until his approval has been given, not to be adopted for work. This order modified the order in council of July 21, 1903.

The amendments to the specifications and Mr. Cooper's letter relating thereto are attached to the order in council and are dated June 2, 1903.

Upon Mr. Cooper receiving a copy of the second order in council he states, in a letter of August 21, to Mr. Hoare: 'I think under fair and broad-minded interpretation, this will allow us to go on and get the best bridge we can, without putting metal where it will be more harm than good.'

This arrangement left the matter of the specifications entirely in the hands of Mr. Cooper, subject only to the approval of the government authorities.

Mr. Cooper, in his evidence, says: 'I assume the full responsibility for the change in the specifications and for the selected unit stresses.' He interpreted the authority given to him as being complete, and the work was carried out using his amendments of the specifications.

Up to the date of the passing of the Guarantee Act, of October, 1903, the Phœnix Bridge Company held to the position expressed in Mr. Reeves' letter of June 19, which

was attached to the contract of the same date with the Quebec Bridge Company. It was not until March 15, 1904, that Mr. Reeves states (Ex. 113E), that they are proceeding with the work vigorously, this letter being in reply to one from Mr. Parent under date of February 22, 1904 (Ex. 113A). In this correspondence Mr. Parent advised Mr. Reeves of the satisfactory financial condition of his company, and the Phœnix Bridge Company felt confident in proceeding with the actual work, knowing, as it did, that payment was certain. The undertaking had now entered into its final stage.

Mr. Scheidl in his evidence (see evidence) refers to certain preliminary work on plans having been done in January, February and March, 1902. A period of inactivity followed, as Mr. Scheidl further states that after the receipt of the revised specifications 'preliminary work' showing practically final results, commenced in July, 1903.

Prior to the date of the contract between the Quebec Bridge Company and the Phœnix Bridge Company, June 19, 1903, the Phœnix Iron Company, who manufacture all the bridge work for the Phœnix Bridge Company, were not equipped to undertake the work. In anticipation of having to do the work they, in the fall of 1902, made additions to their main bridge shop and other improvements in their works. In 1903 they added some heavy machinery to their shops and otherwise improved their works, so as to enable them to manufacture the Quebec bridge for the Phœnix Bridge Company; those were general improvements to their property. Subsequent to June 19, 1903, Mr. Norris, the manager of the works, was instructed to obtain whatever machinery and tools were needed.

HISTORY OF CONTRACTS.

The commission has examined the various contracts and agreements made between the Quebec Bridge and Railway Company and the Phœnix Bridge Company, but finds nothing in them that has direct connection with the cause of the disaster. We give, therefore, simply an historical statement concerning these agreements, but desire to draw attention to the fact that the agreement of April 12, 1900, the agreement of December 19, 1900 (Exhibit 13), and the contract of June 19, 1903 (Exhibit 16), which is an amplification of the first agreement, are, under existing circumstances, of great importance. We recognize that we are not called upon to discuss these agreements from a legal standpoint.

The Phœnix Bridge Company was requested to tender in September, 1898, for the construction of the Quebec bridge (Ex. 6).

According to Mr. Deans (Deans to Hoare, April 14, 1899, Ex. 75-D), there was an understanding at the time that the contract would be awarded to the lowest tenderer.

The following is the letter referred to:—

April 14, 1899.

(Personal and private).

Mr. E. A. Hoare,
Chief Engineer, Quebec Bridge Company,
Quebec, Quebec.

Dear Mr. Hoare,—Mr. Szlapka and I were with Cooper the greater part of yesterday, and you will be glad to learn there was not a single vital or important criticism or mistake found in our plans. All the slight differences, such as dead load, anchor arms, reverse stresses, in one or two members, thickness of some detail plates, &c., were all thoroughly discussed and satisfactorily settled, and not a single one would *affect in any way our price or our proposition.* It was especially gratifying for us to learn this.

Mr. Cooper, however, somewhat upset me, by making the following remark, which of course I understood was entirely personal and without any full knowledge of the situation. He said: 'Well, Deans, I believe that all of the bids will probably overrun the amount which the Quebec Bridge Company can raise, and that the result will be as is usually the case, that all of the bids will be thrown out and a new tender asked on revised specifications and plans.'

I told Mr. Cooper that while this might be the usual procedure, that in the present case it was distinctly understood that whoever was the lowest bidder under the present specifications and plans would be awarded the work, and *if any modifications were made their bid would be altered accordingly*, as this could readily be done through a conference with the Bridge Company's engineers and ourselves; as we could undoubtedly build as cheap a structure as any other company, and that unless this plan was carried out as understood and agreed upon, the present bidders would be placed in a very unfair position after the expenditure of great time and expense.

I finally succeeded in convincing Mr. Cooper that this was the only fair method, but I think it will take the greatest care on your part to see that his report is not worded in such a way as to give the directors an opportunity of following this suggestion. Mr. Cooper undoubtedly desires to be perfectly fair, but not having been through this whole matter like ourselves, does not fully understand the situation. I trust, therefore, that you will give his report the most careful scrutiny, and get it in the right shape before it is submitted, as far as this suggestion is concerned. It would simply be just what our competitors, and particularly the Dominion Bridge Company, would like, or the Union Bridge Company, in fact, and I shall be much interested to hear from you on this point.

You have not advised me to whom I shall send the revised price; including delivery of the material from Quebec and Lévis to site.

Mr. Lindenthal and I have an appointment with Mr. Cooper next Tuesday to discuss the suspension plan.

Kindly advise me when you will desire the revised propositions of the suspension design.

I remain,
Yours truly,

JNO. STERLING DEANS.

On March 1, 1899, the Phœnix Bridge Company handed in its tender, making a lump sum bid as requested. The wording of the tender which was drawn up by the Quebec Bridge Company is as follows:—

'The whole in accordance with sections and specifications shown for substructure and superstructure and such other plans submitted with this tender, which may be adopted by the Bridge Company; for the total sums of money herein stated, &c.'

Mr. Deans wrote in the letter accompanying the tender, as follows:—

'It might be possible, if found necessary or desirable, to make modifications in the requirements which could reduce the cost without materially affecting the efficiency of the structure, and at the proper time we would be glad to discuss this question with your engineers.'

All tenders and plans were handed over to Mr. Cooper for examination and report, after the agreement between that gentleman and the officers of the Quebec Bridge Company had been made on March 23, 1899 (Ex. 112).

On May 8, 1899, and again on May 9, Mr. Deans, at the request of Mr. Hoare, supplemented the Phœnix Bridge Company's bid by letters to Mr. Cooper.

On June 23, 1899, Mr. Cooper reported in favour of the Phœnix Bridge Company's plan and tender (Ex. 9). Tenders were open for acceptance until September 1, 1899.

On August 22, 1899, the directors of the Quebec Bridge Company passed a resolution awarding the contract for the substructure to Mr. Wm. Davis & Sons, and

that for the superstructure to the Phœnix Bridge Company on condition that the contractors accept subsidies and securities in payment.

The lump sum prices are mentioned in the resolution, but with this qualifying clause 'the whole subject to the modifications in the specifications, either decreasing or increasing, or any other made by the company's engineer in the size, depths and locations of the piers and their caissons, at schedule prices in tender submitted.' Apparently this clause changed the contract from a lump sum basis to a unit price basis, as the company's engineer made many modifications. These modifications could not have been avoided and arose mainly from the insufficiency of the plans and the preliminary work done by the Quebec Bridge and Railway Company.

The following letters written at this time made clear the understanding between the two companies and outline an arrangement for settling unit prices which was afterwards adopted for all the Phœnix Bridge Company's contracts:—

QUEBEC, August 22, 1899.

Mr. E. A. HOARE,
Chief Eng'r, the Quebec Bridge Company,
Quebec, Canada.

DEAR SIR,—At the request of the president of the Quebec Bridge Company I hand you in trust to-day the prices we used for plain structural material in our proposal of March 1, '99, for the construction of the Quebec bridge. These figures will fix the basis of comparison when work is ordered ahead as arranged in letters passed between the Quebec Bridge Company and the Phœnix Bridge Company to-day. You will notice these prices are higher than figures ruling on March 1, '99,—lower than those ruling to-day. Plates and shapes 1·80 c. per pound.

Steel castings in rough 3·50 c. per pound.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

QUEBEC, August 23, 1899.

Mr. E. A. HOARE,
Chief Engineer, Quebec Bridge Company,
Quebec, Quebec.

DEAR SIR,—Referring to the figures handed you to-day, you are at liberty to show same to the Hon. S. N. Parent, president of the Bridge Company, for his personal information. I feel certain a knowledge of these figures will not be allowed to go further, or be used against our interests, otherwise I would not be justified in giving out same.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

QUEBEC, August 23, 1899.

Mr. JOHN STERLING DEANS,
Chief Engineer, Phœnix Bridge Company.

DEAR SIR,—Referring to yours of this day, I beg to state that this company is ready to enter into a contract with your company for the superstructure of our proposed bridge, subject to the modifications in the specifications either decreasing or increasing, or any other that may have to be made in size, depths and locations of the piers and their caissons; provided you accept in payment your share of the amount of $1,500,000 in subsidies or their equivalent, and the difference in bonds

given in trust as collateral security, the value and interest on same, at their redemption on conditions to be agreed upon, but at any rate the company will decide before the bridge is open for traffic to redeem the said bonds at face value or surrender them to the contractors; this company binding themselves to transfer you your proportionate share of any further subsidies or guarantees of interest that they may receive towards the construction of the said bridge. We will furnish by an early mail a statement showing the position of the company, its available subsidies and prospects as to resources and earning powers. If your company accepts the above conditions, we on the other hand will accept the condition stated in your letter of this day, that we may order the work from you at any time within two years, providing at the time the work is ordered to proceed either party to the contract may request the prices for plain structural metal revised to agree with the ruling price of metal at that time, and provided also that you give us to-day the price of your metal on which you have based your tender. This option is open for fifteen days from this date.

Yours truly,

S. N. PARENT,
Pres., Q. B. Co.

QUEBEC, CAN., August 23, 1899.

Hon. S. N. PARENT,
President, the Quebec Bridge Company,
Quebec, Canada.

DEAR SIR,—In our letter of March 1, 1899, handing you our proposal for the construction of the Quebec bridge, we stated, 'proposal to be accepted and *work ordered to proceed* on or before July 1, 1899'; later on the time was extended to September 1, 1899. Now, as you do not find it possible to order the work to proceed before September 1, 1899, we will adhere to the terms of our proposal, and upon receipt of the statements promised, take up the question of financing; extending to the Quebec Bridge Company the privilege of ordering the work ahead at any time in the near future, say one or two years; providing at the time the work is ordered to proceed either party to the contract may request the prices for *plain* structural metal revised to agree with the ruling price of metal at the time. I feel quite certain upon carefully considering this matter, you will see that this is a very reasonable proposition. We do not benefit a dollar; our profit remains as in our original proposal and all other items, but the one item mentioned. I hope to receive your favourable reply to-day, when I am sure we will be able to interest our friends to assist in the financing of the enterprise.

Yours truly

JNO. STERLING DEANS,
Chief Engineer.

The Phœnix Bridge Company declined to accept the securities of the Quebec Bridge Company in payment for work, but made a strong effort on behalf of the Quebec Bridge Company to place those securities with certain American financial firms of high standing. This effort did not succeed, the reason for the failure being given by Mr. Deans in his testimony (see evidence), and, briefly put, was that the financiers said there was not sufficient traffic and revenue in sight to justify the investment.

During the first two weeks of April, 1900, correspondence was in progress concerning the lengthening of the main span.

On April 5, 1900, the directors of the Quebec Bridge Company appointed committees to conclude arrangements with the contractors both for substructure and for superstructure.

On April 12, 1900, one committee met Mr. Deans in Mr. Cooper's office in New York, and awarded to the Phœnix Bridge Company the contract for the entire superstructure, the terms and conditions of this award being set out in the agreement of even date as follows:—

New York, April 12, 1900.

It is hereby agreed between the Quebec Bridge Company, represented by the Hon. S. N. Parent, president, of the first part, and the Phœnix Bridge Company, represented by John Sterling Deans, chief engineer, of the second part, as follows:—

To wit: That the party of the first part does hereby award the contract for the construction of the superstructure and steel anchorages of the bridge to be built over the river St. Lawrence, near Quebec, to the party of the second, upon the cash price tendered on March 1, 1899, subject, however, to modifications as to base price of metal stated in letter addressed to E. A. Hoare, company's engineer, dated August 23, 1899, and endorsed by said engineer, the superstructure and steel anchorage to be ordered within three years from date of this present agreement.

The party of the second part hereby agrees to deliver complete all steel required for both anchorages at the respective pier sites within four months after approval of detail plans of same.

The price to be paid for the said metal anchorages by the party of the first part will be fixed at the rate to be mutually agreed upon at the date that the metal is ordered, on delivery at bridge site as aforesaid in good condition, in cash, payable in monthly estimates, less 20 per cent drawback until the anchorage piers are complete, the party of the first part undertaking to pay all custom charges.

The party of the second part hereby agrees to complete all the general and detail plans for the entire superstructure with all possible speed, and to furnish the details of the metal anchorages by the 15th day of June, 1900, and to furnish any other data required by the engineer for arranging dimensions of bridge seats and foundations.

It is further understood that the party of the first part is to have the privilege of ordering the superstructure in whole or any complete portion of the structure at any time within the said three years. It being, however, agreed that the party of the second part is to have the order for whole or any portion at least six months in advance of time said whole or portion is to be ready for erection.

The price of metal now used for the steel anchorages as above is not to be a basis for the price of the remaining metal of superstructure. The price of metal is to be mutually agreed upon at the time each portion of the structure is ordered, according to letter dated August 23, 1899, aforesaid.

It is further agreed that this agreement shall not take effect until approved by the board of directors of Quebec Bridge Company and Phœnix Bridge Company, respectively.

S. N. PARENT,
Pres., Quebec Bridge Co.

JNO. STERLING DEANS,
Chf. Eng., the Phœnix Bridge Co.

On April 14, 1900, Mr. Deans wrote to the Hon. Mr. Parent, asking if the board had approved the agreement of April 12, and stating his understanding of the respective powers of Messrs. Cooper and Hoare. He asked Mr. Parent to confirm this understanding.

(Exhibit No. 75-K.)

April 14, 1900.

Hon. S. N. PARENT,
Pres., Quebec Bridge Company,
Quebec, Canada.

DEAR SIR,—In view of the extreme importance of avoiding delay on your work, which we all appreciate, I write to ask you to kindly wire us when our recent agreement has been approved by your board and they have decided to order the metal work of anchorages.

We understand that in all engineering matters, we are to receive our instructions from Mr. E. A. Hoare, your engineer, and that he works under authority from your board. Please advise if we are correct in this.

Further, we understand that all of our detailed plans of the structure, including sections, &c., must have the approval of Mr. Theo. Cooper, consulting engineer, 35 Broadway, New York, N.Y. Please advise us if we are correct in this.

I write you on these matters in advance of receiving your instructions to proceed, that there may not be the least delay in knowing how to proceed.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

On April 19, 1900, the directors of the Quebec Bridge Company approved the agreement of April 12, but subject to the condition that it was not to take effect until the agreement with Mr. Davis should be concluded.

On April 21, 1900, the Hon. S. N. Parent wired Mr. Deans, in answer to his letter of the 14th inst., as follows:—

April 21, 1900.

J. S. DEANS,
Phœnix Bridge Company,
Phœnixville, Pa.

Agreement made in New York April 12, approved by board. Proceed with plans immediately so as to enable us to order steel for anchorage piers upon approval of same. Arrangements made with Davis. You can confer with Cooper and Hoare *re* plans.

S. N. PARENT,
Pres., Q. B. Co.

On the same day Mr. Barthe wrote to Mr. Deans inclosing a copy of the minute of the resolution of the board of directors, confirming the agreement of April 12, and also confirming the Hon. S. N. Parent's telegram of that date.

QUEBEC, April 21, 1900.

Letter headed Quebec Bridge Co.

Mr. J. S. DEANS,
Phœnix Bridge Company,
Phœnixville, Pa.

DEAR SIR,—I am instructed to confirm you the telegram which was sent this morning by the president, as follows:—

April 21, 1900.

J. S. DEANS,
Phœnix Bridge Company,
Phœnixville, Pa.

Agreement made in New York April 12, approved by board. Proceed with plans immediately so as to enable us to order steel for anchorage piers upon approval of

same. Arrangements made with Davis. You can confer with Cooper and Hoare *re* plans.

S. N. PARENT,
Pres., Q. B. Co.

I also beg to inclose copy of resolution adopted by the board of directors this morning.

Yours truly,

ULRIC BARTHE,
Secretary.

On May 5, 1900, the directors of the Quebec Bridge Company passed a resolution changing the main span from 1,600 to 1,800 feet, and directing the engineers of the company to instruct the contractors to prepare plans accordingly.

On June 15, 1900, Messrs. Cooper, Hoare and Deans met in New York, and agreed on the price to be paid for the anchorage metal, this price being fixed in accordance with the terms of Mr. Deans' letter of August 23, 1899.

On December 19, 1900, a further agreement in accordance with the terms of the agreement of April 12, 1900, was made for the construction of the approach spans.

Revised Agreement.

Dated, New York, Dec. 19, 1900.

It is hereby agreed between the Quebec Bridge Company, represented by the Hon. S. N. Parent, president, party of the first part, and the Phœnix Bridge Company, represented by John Sterling Deans, chief engineer, party of the second part, as follows:—

The party of the second part agrees to deliver and erect complete, according to specifications hereto attached, forming part of these presents, all the steel work required for both the approaches of the proposed bridge over the St. Lawrence river at Quebec, within six months after the approval of detailed plans by the engineers of the party of the first part, which shall allow final delivery of this metal work to be made not later than September 1, 1901.

The party of the first part agrees to pay to the party of the second part for said metal approaches at the rate of 4·114 cents per pound erected and painted complete, in cash, upon the certificates of the engineer of the party of the first part and the Dominion government and provincial engineer, and the engineer of the city of Quebec, of the erection of each approach.

Should the metal work of either of the approaches not be erected on or before January 1, 1902, due to causes beyond the control of the party of the second part, then the party of the second part shall be paid in cash not later than January 15, 1902, on account of the metal work delivered at the bridge site, 3·314 cents per pound, less 20 per cent reserved until the metal work is erected. If either of the approaches is not erected before January 1, 1903, due to causes beyond the control of the party of the second part, then the party of the first part agrees to pay to the party of the second part the 20 per cent reserve in cash not later than January 15, 1903.

It is further understood that the party of the first part shall benefit to the extent of any drop in the base price of metal between the date of this agreement and May 1, 1901, said drop in the base price of metal to be determined as per agreement for anchorage metal, dated April 12, 1900.

It is further understood the party of the first part shall pay all custom duties and charges.

The price of metal now used for the steel in approaches as above, is not to be a basis for the price of the remaining metal of the superstructure. The price of metal

for the said remaining superstructure is to be mutually agreed upon at the time each portion of the superstructure is ordered, according to letter dated August 23, 1899, aforesaid.

It is further agreed that this agreement shall not take effect until approved by the board of directors of the Quebec Bridge Company and the Phœnix Bridge Company respectively.

S. N. PARENT,
President, Quebec Bridge Company.

JNO. STERLING DEANS,
Chief Engineer, the Phœnix Bridge Company.

The agreement was confirmed by the board of directors on January 17, 1901, and this contract was carried out by the Phœnix Bridge Company during 1901, 1902 and 1903.

In the spring of 1903, the question of the main span was taken up, and on June 19, 1903, the final contract was entered into. This is in accordance with Mr. Deans' letter of August 23, 1899, and with the terms of the award of April 12, 1900. The contract reads as follows:—

ARTICLES OF AGREEMENT made and concluded this 19th day of June, 1903, between the Quebec Bridge Company (Limited), a corporation of the province of Quebec, Canada, party of the first part, and the Phœnix Bridge Company, a corporation of the State of Pennsylvania, party of the second part, witnesseth:

First.—That the party of the first part does hereby confirm the award (heretofore made) of the contract for the construction of the entire superstructure of the bridge over the River St. Lawrence, near Quebec, in accordance with the plans and specifications hereto attached and made a part thereof, to the party of the second part, for the cash prices named in schedule paragraph (6).

Second.—That the party of the first part agrees to pay all custom duties, entry fees and expenses, on materials and plant.

Third.—That for and in consideration of the payments and covenants to be made and performed by the party of the first part, the party of the second part does hereby agree to construct, deliver and erect in the most substantial and workmanlike manner, to the satisfaction and acceptance of the consulting engineer and the engineer of the party of the first part, and in accordance with the general plans and specifications hereto attached, and made a part of this agreement, the metal superstructure, railings, screens and guard rails, also the timber for tracks and highway floors, of the bridge over the St. Lawrence river, near Quebec, consisting of one central span of eighteen hundred feet and two side or anchor spans of five hundred feet each.

Fourth.—That before any work is done under this agreement the detailed plans shall be approved by the engineers of the party of the first part and the chief engineer of the Department of Railways and Canals of the Dominion of Canada.

The engineer of the party of the first part or his duly appointed representative shall have the right to inspect all material covered by this agreement, at all stages of the work, and shall have full power to condemn or reject any work or material of inferior quality and not in strict accordance with the requirements of this agreement.

Fifth.—The said superstructure shall be completed by the 31st day of December, 1906, unless delayed or prevented by strikes, floods, or other causes beyond the control of the said party of the second part, or unless the party of the first part shall fail to make any of the payments as hereinafter stipulated or to keep any of its covenants herein contained.

The above date of completion is based upon the understanding that work under this agreement may proceed uninterruptedly from this date.

Sixth.—In consideration of doing and performing the work embraced in this agreement, the party of the first part hereby covenants and agrees to pay to the party of the second part, in addition to all custom duties, entry fees and expenses, as provided in paragraph (2), the following prices, namely:—

			Estimated Quantities.
Steel in trusses and bracing complete......	per lb.	5·60c.	50,897,000 lbs.
Steel in floor beams and stringers complete..	"	5·35c	7,700,000 "
Steel in railings, screens and guard rails complete..	"	5·55c.	755,000 "
Steel in washers, bolts, &c., complete......	"	5·75c.	120,000 "
Timber in railway track in place complete	per M. ft. B.M	$35	865,000 ft.
Timber in highway floors in place complete	per M. ft. B.M.	$33	725,000 ft.

Payment shall be made in the following manner, to wit:—

On or about the last day of each month, during the progress of this work, the engineer of the party of the first part shall estimate the value of material furnished and work done at the manufactory of the said party of the second part at Phœnixville, Pa., also material delivered at bridge site and work done at bridge site at the schedule rates hereinafter specified for the several classifications, and ninety per cent of the amount of said estimates shall be paid in cash to the party of the second part on or before the tenth day of the following month. After the ten per cent reserve amounts to a total of one hundred thousand dollars ($100,000) the monthly estimates thereafter shall be paid in full. The balance due to said party of the second part shall be paid in cash to it in thirty days after all the work embraced in this contract is completed in accordance with the plans and specifications and accepted by the engineer of the party of the first part, and only after the bridge has been tested with the specified loads or in any other manner required by the engineers of the party of the first part, and has obtained certificates from the chief engineer of the Department of Railways and Canals of the Dominion of Canada, stating that the bridge has been accepted and can be safely used for railway and highway traffic. It is agreed that the absolute title to all material at Phœnixville, or elsewhere, ninety per cent of the value of which has been included in any monthly settlement, shall upon payment pass to the party of the first part, and the party of the second part will deliver a bill of sale therefor to the party of the first part.

Seventh.—The schedule rates to be used in making the monthly estimates for the work as it progresses are as herein stated. If there are any other items than those here indicated, the schedule rates are to be determined by the engineers of the party of the first part.

Classification.	Trusses and Bracing.	Floor Beams and Stringers.	Railway, Screen and Guard Rails.	Washers, Bolts, &c.
	$ cts.	$ cts.	$ cts.	$ cts.
Metal rolled at mills (including approved design and detail drawings)................	2 55	2 55	2 55	2 55
Metal manufactured at shops................	3 60	3 35	3 55	3 75
Metal delivered............................	4 10	3 85	4 05	4 25
Metal erected..............................	5 45	5 20	5 40	5 60
Metal erected and painted, complete.........	5 60	5 35	5 55	5 75
Timber in railway track....................	Delivered at site, $28 per M. feet board measure.			
" "	In place, complete, $35 per M. feet board measure.			
Timber in highway floors...................	Delivered at site, $26 per M. feet board measure.			
" "	In place, complete, $33 per M. feet board measure.			

Eighth.—The party of the second part shall take, use, provide and make all proper, necessary and sufficient precautions, safeguards and protections against the occurring or happening of any accidents, injuries, damages, or hurt to any person or property during the progress of the erection of the work herein contracted for, and indemnify and save harmless the said party of the first part, from the payment of all sums of money by reason of all or any such accidents, injuries, damages or hurt that may happen or occur upon or about said work, and from all fines, penalties and loss incurred for or by reason of the violation of any city or borough ordinance or harbour regulations, or laws of the Dominion of Canada or province of Quebec, for which they are responsible, while the said work is in progress of construction.

Ninth.—It is understood and agreed that the party of the second part shall indemnify and protect the party of the first part from all claims under any law for labour and materials furnished under this contract, and shall furnish the said party of the first part with satisfactory evidence when called for that all persons who have worked for or furnished materials to the contractor or sub-contractors have been fully paid or satisfied, and failing which an amount necessary and sufficient to meet the claims of the persons aforesaid shall be retained by the party of the first part from any moneys due said party of the second part until the liabilities aforesaid have been paid; this clause is not intended, however, to apply to claims made against the party of the second part which he *bona fide* contests his liability for, and when the work is completed the party of the second part will furnish the party of the first part with a satisfactory bond indemnifying the party of the first part from all and any of the claims that may be against them by reason of any acts of the party of the second part or sub-contractors.

Tenth.—All materials and supplies put on the work and settled for through progress estimates in the manner provided for in this contract shall become the property of the party of the first part.

Eleventh.—The party of the second part shall conform to all Harbour Commissioners' regulations for the safety of vessels when passing the bridge site, and the party of the second part shall further be responsible for all damages to vessels that may arise from neglect or proper precautions, or damages to the work in progress from any cause until the entire superstructure is completed and accepted by the party of the first part and the Government of the Dominion of Canada.

Twelfth.—The party of the second part shall restore at his own cost all or any part of the work that may be damaged or destroyed before its acceptance by the aforesaid parties, notwithstanding that payments on account of progress estimates may have been made previous to the occurrence of such damages.

Thirteenth.—The party of the second part further agrees that the whole of the working plant to be placed and used by him on the bridge superstructure, including all mechanical appliances, hoisting machines, motive power, tools, machinery and equipment, used in said work, and buildings, workshops, landings or false works erected for the purpose of the present contract, shall be and remain the property of the party of the first part until the completion of the works, as a guarantee of the due and proper execution of the works.

Fourteenth.—The party of the second part will be obliged to give a guarantee company bond satisfactory to the party of the first part, amounting to one hundred thousand dollars ($100,000), which, together with the one hundred thousand dollars ($100,000) reserved according to the sixth clause hereof, shall constitute a fund of two hundred thousand dollars ($200,000) as a guarantee for the faithful performance of the work under this agreement.

Fifteenth.—The price of extra work cannot be claimed by the party of the second part unless same has been authorized in writing by the engineer and approved of by a resolution of the board of directors of the party of the first part.

Sixteenth.—The decision of the engineers of the party of the first part shall control as to the interpretation of the plans and specifications attached and the work

performed under this agreement, during the execution of the work, but if either party shall deem itself to have been aggrieved by any decision, it may require the dispute to be finally and conclusively settled by the decision of three arbitrators, the first to be appointed by the party of the first part, the second by the party of the second part, and the third to be appointed by the first two named. By such decision both parties hereto shall be finally bound, it being understood that no such submission to arbitration shall suspend or postpone the making of any of the payments as herein provided, except only to the extent actually involved therein.

In witness whereof the said parties to these presents have hereunto set their respective corporate seals. Dated the day and year first herein written.

THE PHŒNIX BRIDGE COMPANY.
By DAVID REEVES, (Seal)

Attest:
GEORGE GERRY WHITE,
Secretary.

(Seal Q. B. Co.)
ULRIC BARTHE,
Secretary Treasurer.

S. N. PARENT,
President.

This agreement was confirmed by the directors of the Quebec Bridge Company on the day that it was made.

Its acceptance by the Phœnix Bridge Company was only provisional, Mr. Reeves attaching the following letter to the signed agreement:—

PHŒNIXVILLE, PA., June 19, 1903.

HON. S. N. PARENT,
President, Quebec Bridge Company, Limited,
Quebec, Canada.

DEAR SIR,—We hand you herewith articles of agreement for the construction of the superstructure of main spans of the Quebec bridge, executed by this company, upon the understanding that said agreement shall not become operative until the legislation proposed at present session of parliament has taken place and the financial arrangements insuring payments of estimates under said agreements have been arranged to the satisfaction of this company, and letters have passed between the two companies to this effect.

In the meantime, that there may be the least delay, we agree to proceed with all possible speed with the stress sheets and detailed drawings, as soon as the revised specifications have been furnished to us, approved by the government engineers.

It is further to be understood, that the time named in the agreement for the completion of the work is one which we do not guarantee, and it is based upon the work proceeding uninterruptedly from this date. The date named we will do our best to keep. We cannot accept any responsibility for damages of any kind which may result from any delay in the completion beyond the date fixed in agreement.

We agree, however, to complete the work under the terms of said agreement by December 31, 1908, and will pay to the Quebec Bridge Company, Limited, $5,000 per month for each month thereafter that the work called for by the said agreement is not completed.

Should there be any stoppage of the work for a period of six months from any cause for which the Phœnix Bridge Company is not responsible, except from strikes and floods, thereupon an estimate shall be made of the total expense incurred by the Phœnix Bridge Company on account of said agreement to date, and after deducting all payments made to date, the balance plus ten per cent of said total expense, shall be immediately due and paid to the Phœnix Bridge Company in cash by the Quebec Bridge Company, Limited.

We agree to modify the prices made in this agreement, to the extent of any variation in the base price of plain metal on cars Philadelphia, from $1.80 per pound,

and which variation may occur between this date and August 15, 1903; said change, if any, to be agreed upon by your chief engineer, Theo. Cooper, consulting engineer, and John Sterling Deans, chief engineer of this company.

It is moreover understood that the agreement shall not be assigned or transferred by either party to the same without the consent of the other.

The articles of agreement handed you herewith shall become binding only upon my receipt from you of a duplicate duly executed by your company, accompanied by a letter confirming the understanding as expressed above.

Yours truly,

DAVID REEVES,
President, the Phœnix Bridge Company.

On February 22, 1904, the Hon. S. N. Parent wrote to Mr. Reeves as follows:—

QUEBEC, February 22, 1904.

DAVID REEVES, Esq.,
President, Phœnix Bridge Company,
410 Walnut Street.

DEAR SIR,—Referring to the contract between the Quebec Bridge Company (now styled the Quebec Bridge and Railway Company) and your company, and also to the letters exchanged between our companies in last June, and particularly to the first clause in your letter of June 19, 1903, I beg to inform you that the legislation proposed in the last mentioned letter has taken place, and that the following financial arrangements insuring payments of estimates under this company's agreement with you have been made, namely:—

1. Provision has been made for the payment and discharge of the outstanding bonds and mortgages of the Quebec Bridge Company referred to in section 10 of the Act of Parliament, 3 Edward VII., chapter 177, in accordance with the terms of that section.

2. The agreement in reference to the government guaranty referred to in section 13 of the same Act was, on the 28th day of January, submitted to and approved by a general meeting of the shareholders of this company duly called for that purpose in accordance with the provisions of that section.

3. This company has arranged with the present subscribers to the capital stock of the company for the surrender of the same in accordance with clause 3 in the agreement set forth in the schedule to the Act of Parliament (3 Edward VII., chapter 54).

4. Subscriptions have been procured for additional stock of this company to the amount of $200,000 as provided for in clause 4 of the last mentioned agreement.

5. Arrangements have been made for underwriting the bonds referred to in the fifth and sixth clauses of the said last mentioned agreement as issued.

6. The stockholders and board of directors of this company have duly performed everything required by the two Acts of Parliament and the said agreement, as conditions precedent for a compliance with the terms imposed upon this company by the aforesaid agreement.

It is of course understood that the change of name of the Quebec Bridge Company to that of the Quebec Bridge and Railway Company shall not in any way impair, alter or affect the rights or liabilities under the contract entered into with your company in June, 1903.

Truly yours,

S. N. PARENT,
President.

On March 15, 1904, Mr. Reeves wrote to the Hon. S. N. Parent, advising him that the assurances contained and the terms expressed in his letter of February 22 were satisfactory to the Phœnix Bridge Company (Exhibit 113 C).

On March 17, 1904, Mr. Deans wrote to Mr. Parent, stating that the contract is now closed, and congratulating Mr. Parent upon his success.

There are no subsequent alterations of these business arrangements.

The Phœnix Bridge Company had not completed the work under this contract when the accident took place on August 29, 1907.

The connection of the government with the enterprise provided the means for building the bridge, and the final approval of plans rested with it, but in no way did the government exercise any check on the work itself, or any authority over the contractors. The administration of the contract and the disposition of the funds supplied by the government were left entirely in the control of the Quebec Bridge Company, subject to the approval of the estimates by the government inspector, and except that the quantities of material were checked at Phœnixville by a clerk appointed by the Department of Railways and Canals, and an officer of that department visited the bridge in connection with the checking of estimates, there was no supervision on the part of the government.

By no act did the government assume or exercise authority over the Phœnix Bridge Company, nor did it intervene under the contract for the bridge; the checking and inspection done by the government and above referred to were with reference to the operations of the Quebec Bridge Company, as the agreement for financing was between the government and the Quebec Bridge Company. The only party, therefore, who was competent to deal with the Phœnix Bridge Company, and who only did deal with it, was the Quebec Bridge Company.

On the part of the government, its confidence in the Quebec Bridge Company was complete; in so far as the integrity of the structure itself was concerned, this was because of the presence of Mr. Cooper as the consulting engineer for the Quebec Bridge Company. The government was familiar with the terms of the contract between the two companies.

HENRY HOLGATE,
Chairman.

J. G. G. KERRY,
J. GALBRAITH.

APPENDIX No. 4.

THE PHŒNIX BRIDGE COMPANY.

This company was incorporated under the authority of an Act of the General Assembly of the Commonwealth of Pennsylvania, entitled 'An Act to provide for the incorporation and regulation of certain corporations,' which received approval on April 29, 1874.

The date of the letters patent incorporating the Phœnix Bridge Company is April 2, 1884, the original shareholders being David Reeves, William H. Reeves, Adolphus Bonzano, George Gerry White and Carrol S. Tyson.

The Phœnix Bridge Company was formed, according to its charter (Exhibit 119), 'for the purpose of manufacturing articles of commerce from iron and steel, and the building of bridges, roofs, viaducts and all kinds of structural work from metal or wood, or both, and to erect and construct such improvements and erections as they may deem necessary, and in general to do all such other acts and things as a success-

ful, convenient prosecution of said business may require and as may be necessary, incidental and appurtenant thereto. The business of the company to be transacted in the borough of Phœnixville, county of Chester, in this commonwealth.'

The company's charter is a perpetual one. The capital of the company is $100,000, divided into shares of $100 each.

The Phœnix Bridge Company is an engineering and contracting company, and is not a manufacturing company. It has an arrangement with the Phœnix Iron Company, an entirely independent corporation, under which the material for its bridges and other structural work is manufactured and fabricated in accordance with the Bridge Company's instructions. The financial control of both companies is the same, but formal methods of accounts, charges and payments are maintained between the two companies precisely as in other contracts that either company might enter into. This arrangement has been in force since 1884, and much of the material for the Quebec bridge was manufactured and all was fabricated by the Phœnix Iron Company to the order of the Phœnix Bridge Company in accordance with this arrangement.

The Phœnix Bridge Company is a tenant of the Phœnix Iron Company at Phœnixville, and pays rental to it for office buildings, &c.

Delivery is made to the Phœnix Bridge Company as soon as the material is loaded on cars for shipment, and that company attends to its transportation and erection.

In effect, the Phœnix Bridge Company sublet the manufacture of the Quebec bridge to the Phœnix Iron Company, but itself undertook the design and erection. No mention of the Phœnix Iron Company is made in the contract with the Quebec Bridge and Railway Company or in any of the correspondence relating to it.

The officers of the Phœnix Bridge Company and of the Phœnix Iron Company respectively are as follows:—

PHŒNIX BRIDGE COMPANY:

David Reeves, president.
Wm. H. Reeves, general superintendent.
Geo. Gerry White, secretary.
Frank T. Davis, treasurer.
John Sterling Deans, chief engineer.

PHŒNIX IRON COMPANY:

David Reeves, president.
Wm. H. Reeves, general superintendent.
Geo. Gerry White, secretary.
George C. Carson, Jr., treasurer.
Frank P. Norris, manager.

HENRY HOLGATE,
Chairman.

J. G. G. KERRY,
J. GALBRAITH.

APPENDIX No. 5.

THE EFFECT OF FINANCIAL LIMITATIONS UPON THE DESIGN OF THE BRIDGE, AND A DISCUSSION OF THE EVIDENCE RELATING TO THIS.

The fact that the carrying out of the bridge project was for years delayed by lack of funds, being a matter of common knowledge, it was desirable to investigate the effect of this condition upon the design and execution of the work.

Mr. Cooper has stated that 'during the early progress of the work it was an open secret that the Quebec Bridge Company had but a small amount of money in sight.' (See Evidence.)

In proof of this statement reference may be made to the following facts:—

Between 1887 and 1898 the Quebec Bridge Company accomplished practically nothing.

In 1900, it let the contract for the substructure, payment to be made partly out of subsidies and partly in bonds of the company to be accepted at 60 per cent of the face value, and offered its superstructure contract on similar terms.

In 1900 its securities were thoroughly investigated by the leading firms of American bankers, who declined to invest in them.

The Phœnix Bridge Company was paid for the construction of the approach spans not by the Quebec Bridge Company, which ordered them, but by Mr. M. P. Davis. (Deans to Barthe, August 23, 1901, Ex. 74 H.)

It must have been clear to the engineers from the first that the financial conditions were such that nothing but absolutely necessary work could be undertaken.

The effect of the lack of funds is noticeable in the methods of calling for tenders, and of letting contracts, and in the delays that occurred in the execution of the work.

In September, 1898, the bridge contracting firms were asked to submit tenders upon their own designs, to be drawn in accordance with certain specifications. Practically this meant that each bridge company was asked to spend several thousand dollars on the preparation of plans, and that in return it was given an opportunity to bid for a contract to be let by a company of weak financial standing. The result was that although the magnitude of the work placed it outside the limits of established practice, most of the tenders submitted were made from immature studies based upon insufficient data. The evidence shows that the Phœnix Bridge Company gave more time and attention to the competition than any other tenderer, but the error afterwards made by it in assuming the weight of the structure for final designs shows how faulty the estimate accompanying its original tender was. We consider that the procedure adopted in calling for tenders was not satisfactory in view of the magnitude of the work, and was not calculated to produce the most efficient results.

In his evidence (see Evidence) Mr. Hoare ascribes the failure of the Quebec Bridge Company to take advantage of the lump sum tender of the Phœnix Bridge Company to lack of funds. We are satisfied from the knowledge gained during the designing of the 1,800-foot span, that the 1,600-foot span could not have been built with the weight of metal stated in the tender of March 1, 1899. Mr. Deans' letter to Mr. Hoare (Ex. 75 D, April 14, 1899) shows that the Phœnix Bridge Company expected that its tender would be modified before the work was built. The letter is as follows:

April 14, 1899.

(Personal and private.)

Mr. E. A. Hoare,
Chief Engineer, Quebec Bridge Company,
Quebec, Quebec.

Dear Mr. Hoare,—Mr. Szlapka and I were with Mr. Cooper the greater part of yesterday, and you will be glad to learn there was not a single vital or important criticism or mistake to be found in our plans. All the slight differences, such as dead load, anchor arms, reverse stresses, in one or two members, thickness of some detail plates, &c., were all thoroughly discussed and satisfactorily settled, and not a single one would *affect in any way our price or our proposition.* It was especially gratifying for us to learn this.

Mr. Cooper, however, somewhat upset me, by making the following remark, which of course I understood was entirely personal and without any full knowledge of the situation. He said: 'Well, Deans, I believe that all of the bids will probably overrun the amount which the Quebec Bridge Company can raise, and that the result will be as is usually the case, that all of the bids will be thrown out and a new tender asked on revised specifications and plans.'

I told Mr. Cooper that while this might be the usual procedure, that in the present case it was distinctly understood that whoever was the lowest bidder under the present specifications and plans would be awarded the work, and *if any modifications were made their bid would be altered accordingly,* as this could readily be done through a conference with the Bridge Company's engineers and ourselves; as we could undoubtedly build as cheap a structure as any other company, and that unless this plan was carried out as understood and agreed upon, the present bidders would be placed in a very unfair position after the expenditure of great time and expense.

I finally succeeded in convincing Mr. Cooper that this was the only fair method, but I think it will take the greatest care on your part to see that his report is not worded in such a way as to give the directors an opportunity of following this suggestion. Mr. Cooper undoubtedly desires to be perfectly fair, but not having been through this whole matter like ourselves, does not fully understand the situation. I trust, therefore, that you will give his report the most careful scrutiny, and get it in the right shape before it is submitted, as far as this suggestion is concerned. It would simply be just what our competitors, and particularly the Dominion Bridge Company, would like, or the Union Bridge Company in fact, and I shall be much interested to hear from you on this point.

You have not advised me to whom I shall send the revised price, including delivery of the material from Quebec and Lévis to site.

Mr. Lindenthal and I have an appointment with Mr. Cooper next Tuesday, to discuss the suspension plan.

Kindly advise me when you will desire the revised propositions of the suspension design.

Yours truly,

JNO. STERLING DEANS.

We desire to draw attention to this letter, because it indicates that the contract was subsequently awarded on the result of this competition, the basis of the award being a lump sum tender, which could not have been accepted without modifications.

These errors we ascribe to failure on the part of the Quebec Bridge Company to provide for sufficient preliminary studies of the project by its own engineers. It should also be noted that in the opinion of Mr. Cooper the preliminary surveys from which the main spans and the position of foundation piers, &c., were first determined were entirely insufficient (see Evidence); further examinations and borings were made on his advice, and resulted in radical alterations in the design.

In April, 1900, the Phœnix Bridge Company undertook to complete the plans for the bridge with all possible speed. In May, 1900, the Quebec Bridge Company, on the advice of its consulting engineer, determined to adopt a main span of 1,800 feet, and tacitly approved alterations of the specifications. The contractors were ordered to proceed with the designing for the 1,800-foot span under the supervision of Messrs. Hoare and Cooper, but the new specifications, which had to be accepted and officially approved by the Canadian government, were not issued until the summer of 1903. This delay of three years seems to have occurred with the mutual consent of the Quebec Bridge Company and the Phœnix Bridge Company. The Quebec Bridge Company was not in a position to pay for the work, and did not demand that the designing be proceeded with, nor did it furnish the necessary data for the designing. The Phœnix Bridge Company was occupied with other contracts, and did not make any further expenditures on behalf of the Quebec Bridge Company until the financial position was assured.

When the Dominion government finally came to be more closely identified with the Quebec Bridge Company, in 1903, it intimated unofficially to the Phœnix Bridge Company its desire that the bridge should be ready for the Quebec Tercentenary in 1908 (see Ex. 77 U). For this and for ordinary business reasons the Phœnix Bridge Company hurried the work of designing and manufacture as much as possible, this hurry resulting in errors, but not in those errors which were the immediate cause of the accident, these having been previously made. It is necessary in designing a bridge to commence by assuming what its weight will be, and as the design progresses to alter this assumption by calculation from the drawings. In the rush following the final financial arrangements of 1903, the necessity of revising the assumed weights was overlooked both by the engineers of the Phœnix Bridge Company and by those of the Quebec Bridge Company, with the result that the bridge members would have been considerably over-stressed after completion. This error was sufficient to have condemned the bridge had it not fallen owing to other causes.

During the period occupied in the development of the details of the design, the designing engineer and his staff were absorbed in the preparation of detail plans, and this resulted in the slighting of matters of primary importance.

Under the circumstances this condition was unavoidable, but could have been improved had the time between April, 1900, and August, 1903, been used in consideration and preparation of designs; otherwise business matters were in such shape that the Phœnix Bridge Company were not warranted in expending time and money in this direction.

It is also proper to inquire whether the engineers modified their designs to the injury of the bridge on account of the financial conditions.

The importance of economy in the preparation of the first tenders is shown by the letter already quoted.

The tenders, however, had to conform to the original specifications, and there is no evidence of unwise economy in the provisions of these.

Mr. Cooper's attitude with regard to cost, while he was examining the plans and tenders, is shown by the following letter:—

April 19, 1899.

(Personal.)

E. A. Hoare, Esq.,
Chief Engineer, Quebec Bridge Company,
Quebec, Quebec.

Dear Mr. Hoare,—I spent most of yesterday in New York in consultation with Mr. Cooper and Mr. Lindenthal, and found that Mr. Cooper had no serious complaints to make in connection with Mr. Lindenthal's plan; in fact he expressed himself as much interested in the ingenious design.

It developed, however, in conversation, and Mr. Cooper so expressed himself to Mr. Lindenthal, that in view of the amount of the bid under his design, he would not give Mr. Lindenthal's plan careful and detailed consideration, and would so report This rather exasperated Mr. Lindenthal, and for a time I feared he might withdraw his bid, but it was smoothed over and I think will be permitted to stand. Mr. Lindenthal thought that Mr. Cooper should report solely and wholly *on the merits of the several designs,* without any regard to cost, and each design should have the same careful consideration, and that you and your company alone should consider the question of price. I know this is entirely different from Mr. Cooper's instructions, and that it would be useless to spend detailed investigations upon plans which are very expensive in price, but Mr. Lindenthal viewed the matter from an engineer's standpoint, and having taken such unusual pains with the design and estimate felt that he was in a measure being slighted.

Mr. Cooper advises that he will finish about May 1.

I think it of the utmost importance to see you some time before that date, and write to ask if you will not come to New York. Cooper also advised me that he had no authority to receive any revised bids for possible reduction in suspension bridge wire, and I think this entirely proper. It seems to me, however, *that you should have all of these bids in your hands at once, and I will be prepared to submit ours when you come to New York.*

Please let me know at once and by *wire* when you will be in New York.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

In his report upon the competitive tenders submitted on June 23, 1899, Mr. Cooper says:—

'The tender accompanying this plan (from the Phœnix Bridge Company) is the lowest in price, and is the most favourable as to prospective duties upon the materials to be used in its construction. I therefore hereby conclude and report that the cantilever superstructure plan of the Phœnix Bridge Company is the "best and cheapest" plan and proposal of those submitted to me for examination and report.'

There is no evidence whatever to indicate that economy at the expense of efficiency was ever considered by Mr. Cooper. His award was made distinctly to the lowest tenderer, and he so states, but in the preceding paragraphs the accepted design is stated to be 'an exceedingly creditable plan' and 'in accordance with your specifications.'

The full text of the report and Mr. Cooper's evidence show that his award was made for technical reasons, although he did not overlook costs; and he states that (see Evidence) he was left absolutely unhampered in any manner in his report as to which he should consider the best plan and the best bridge.

In a memorandum accompanying his original report, Mr. Cooper indicated his desire to alter the specifications, and to reconsider the length of the main span as soon as proper foundation surveys could be made.

These changes were subsequently made, but it does not appear that economy was the ruling factor in his selections. He unquestionably increased the unit stresses, but not to a point beyond those already adopted by the Bridge Department of the city of New York for its great bridges, and the increase can be stated to be in harmony with the most advanced practice of that time, and due more to an instinct of wise investment than to any endeavour to simply cheapen the structure. The wisdom of his modifications is discussed in appendix 18.

In his evidence (see Evidence) Mr. Cooper has outlined his intentions in making his alterations, and a desire not to involve the Quebec Bridge Company in a greater

expenditure than was at first anticipated is given among them; but on the same page it is sharply stated that he would not recommend any plans that did not promise to give a safe and satisfactory structure.

The facts that have been discussed in this appendix show that while there is no evidence of any cheap and insufficient work being purposely done by either Mr. Cooper or the Phœnix Bridge Company, there is evidence to prove that the financial weakness of the Quebec Bridge Company seriously interfered with the carrying out of the undertaking.

The Phœnix Bridge Company were limited only by the specifications as amended by Mr. Cooper, endorsed by the government and concurred in by themselves, and no sum of money or total weight was set as a limit in the designing or building of the superstructure, the sole aim of all being to produce a safe and economical bridge.

The Phœnix Bridge Company were paid for the work at so much per pound, so there was no incentive to the Phœnix Bridge Company to make the bridge lighter than they deemed it should be.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY,
J. GALBRAITH.

APPENDIX No. 6.

THE HISTORY OF THE DEVELOPMENT OF THE SPECIFICATIONS AND A DISCUSSION OF THE EVIDENCE RELATING TO IT.

During the summer of 1898, Mr. E. A. Hoare, acting in his capacity of chief engineer of the Quebec Bridge Company, prepared the first set of specifications for the construction of the bridge. On July 2, 1898, Mr. Hoare was instructed by resolution of the board of directors of his company to communicate with Mr. Collingwood Schreiber, the deputy minister and chief engineer of the Department of Railways and Canals, so that a set of specifications would be secured that would be satisfactory both to the government and to the Quebec Bridge Company. On direction of Mr. Schreiber, Mr. Hoare submitted his draft specifications to Mr. R. C. Douglas, the bridge engineer of the department, for criticism.

Mr. Douglas states in his evidence (see Evidence) that he read over the specifications with Mr. Hoare, but did not suggest any alterations in them, because Mr. Hoare met his objections by explaining that the specifications would be used only in connection with preliminary competitive tenders and not for the construction of the bridge. He made no official report upon them.

These specifications are, as stated by Mr. Douglas, mainly a direct copy from the general specifications for steel and iron bridges issued by the Department of Railways and Canals in 1896. An examination bears out Mr. Cooper's statement (see Evidence) that they were not drawn by anyone having the magnitude of this bridge structure in mind.

On August 31, 1898, Mr. Schreiber, by letter, notified the Quebec Bridge Company that Mr. Hoare's specifications had been approved (Exhibit 5).

They were printed on order of the Quebec Bridge Company over date of September 1, 1898, and a copy of them was sent out with each of the invitations to tender mailed to bridge contractors in September 1898 (Exhibit 21).

On November 12, 1900, a subsidy agreement No. 13988 (Exhibit 12) was made between the Government of Canada and the Quebec Bridge Company by which, on certain conditions, assistance to the amount of $1,000,000 was promised to the Quebec Bridge Company. The Hoare specifications were made a part of this agreement, with one alteration, viz., the length of the main span was made 1,800 feet instead of 1,600 feet, the Quebec Bridge Company having officially decided on the longer span on May 5, 1900. There is no evidence to show that these specifications were reconsidered at this time by the technical advisers of the government.

The original specifications were not used in the design of the approach spans which were made in 1901-2, alterations being made to meet the wishes of Mr. Douglas, whose approval was required by the deputy minister and chief engineer before payments on subsidy account could be authorized.

Mr. Cooper, in a memorandum accompanying his original report of June 23, 1899 (Exhibit 9), indicated that he thought the specifications could be modified with considerable advantage to the interests of the company. On May 1, 1900, Mr. Cooper recommended to the company the adoption of the 1,800-foot main span, his recommendation being dependent upon the use of certain alterations in the specifications which were, in his opinion, desirable and justifiable. In a letter of even date to the Hon. S. N. Parent, he suggests that he 'be instructed to make such modifications in the adopted competitive plan when adapted to the new lengths, as may tend to reduce the cost without reducing the carrying capacity or the stability of the structure.'

On May 5, 1900, the board of directors of the Quebec Bridge Company directed its engineers (Messrs. Cooper and Hoare) to instruct the contractors (the Phœnix Bridge Company) to prepare plans using the 1,800-foot span recommended by Mr. Cooper. No active effort was made by the officials of either company to carry out these instructions, and the amendments to the specifications which had to be formally approved by the government before the plans could be commenced were not actively discussed until May, 1903. The delay was due to financial reasons, no one knowing when the work would proceed.

The National Transcontinental Railway project, which was made public in the spring of 1903, was so planned that a bridge near Quebec would be a national necessity, and legislation involving a guarantee by the government of the securities of the Quebec Bridge Company was proposed. With the improved financial outlook, the activity of the engineers and contractors was renewed. Mr. Cooper prepared his amendments to the original specifications, and sent them to Mr. Szlapka, the designing engineer of the Phœnix Bridge Company, for his information and criticism. Mr. Szlapka criticized the draft, and returned it to Mr. Cooper, after having taken a copy of it, on May 20, 1903. The comments in his letter show that he had carefully considered the purport of the amendments. Mr. Deans, returning from Ottawa, wrote to Mr. Cooper on May 22, 1903, as follows: 'I was requested by the Ottawa officials to urge upon you to act as promptly as possible in the matter of completing the specifications, and to forward the same to Mr. Hoare without delay. There is urgent necessity of their taking prompt action.'

On May 28, 1903, Mr. Deans wrote to Mr. Cooper, suggesting some alterations in his draft for the amendments, one of which appears in the preface to Mr. Cooper's draft of June 2, 1903. Mr. Cooper completed his draft of the amendments, and forwarded it to Mr. Hoare, accompanied by a memorandum dated June 2, 1903 (Exhibit 21). A copy of the papers was sent also to Mr. Deans.

Mr. Deans, under date of June 4, 1903, acknowledged the receipt of these papers, and expressed the hope that 'we will soon hear that these specifications have been approved by the government.'

On June 16, 1903, Mr. Szlapka, at the request of Mr. Deans, sent to Mr. Hoare two sheets of calculations comparing the stresses permitted under the Hoare specifications with those permitted by the Cooper amendments. In the accompanying letter

(Exhibit 21) he stated: 'With figures given I hope you will be able to see that the difference between the two specifications is very immaterial. Where the new specifications give smaller sections than your specifications, it will be found during actual final computations that owing to the magnitude of the structure, and consequently the very large dead load as compared with the live load, the unit stresses selected are fully justified.' On the sheets accompanying this letter the amendments are referred to as the 'Proposed specifications of June, 1903. (Theo. Cooper and Phœnix Bridge Company.)' The officials of the Quebec Bridge Company were therefore distinctly advised that both Mr. Cooper and the engineers of the Phœnix Bridge Company considered the adoption of the amendments entirely desirable.

Owing to the terms of the subsidy agreement of November 12, 1900 (Exhibit No. 12), it was necessary to have these amendments approved by the government, and they were accordingly transmitted to Mr. Schreiber by the Quebec Bridge Company. Mr. Schreiber handed the papers to Mr. Douglas for report shortly after they reached his office, and on July 9, 1903, Mr. Douglas made his report in writing (Exhibit 63). In it he advised the adoption of many of Mr. Cooper's suggestions, but criticized the high unit stresses that were proposed, and the suggestion made in the memorandum as to using the bridge for heavier rolling loads than those specified in the amendments. He also advised that the Quebec Bridge Company be required to submit new specifications, and not merely amendments to the approved Hoare specifications.

Mr. Douglas' opposition was evidently anticipated, as will be seen by the letter from Mr. Hoare quoted in the evidence. On receipt of the report of July 9, 1903, Mr. Schreiber had to decide whether he would depend upon Mr. Cooper or upon Mr. Douglas for technical advice, and evidently decided in favour of the former, for, as stated in the evidence, Mr. Douglas from that time had no authoritative connection with the undertaking.

Mr. Cooper's intention in making these amendments was, as stated in his evidence, to rearrange the wind and live loadings so that they would more nearly correspond to his own prediction of the actual loadings that would come upon the structure; and accordingly he decreased the wind load and increased the rolling live load. He was also of the opinion that the maximum stresses might safely be increased, and had recommended the 1,800-foot span on the assumption that this increase would be permitted. He was throughout impressed with the necessity of making his changes without adding to the financial demands on the resources of the company.

Mr. Schreiber's views are stated in a letter to Mr. Cooper under date of July, 1903 (Exhibit 21), which reads as follows:—

Dear Sir,—I have received from Mr. E. A. Hoare two memoranda made by you in respect of the plans of the superstructure of the Quebec bridge, suggesting certain modifications which you consider desirable.

Inasmuch as the contract for this structure contains an express specification by which I am bound, I am unable, as matters stand, to sanction any deviations from it.

I am, however, strongly impressed with the expediency, in order not to hinder the progress of the work of manufacture, of permitting you certain latitude in the preparation of the detail plans, even to the extent of adopting (with my own concurrence) such modifications as may appear proper; and holding this view, I have asked that authority be given me by order in council which will enable me to act in that direction.

Nothing can, of course, be done until such order is passed, but on receipt of it I will communicate with you immediately.

Faithfully yours,

Mr. Schreiber communicated with the Minister of Railways and Canals as indicated in the foregoing letter on July 9, 1903, and his recommendations were transmitted by the minister to council on July 18, 1903, and form the substance of the order in council of July 21, 1903 (Exhibit 17). This order reads as follows:—

EXTRACT from a report of the committee of the Honourable the Privy Council, approved by the Governor General on July 21, 1903.

On a memorandum dated July 18, 1903, from the Minister of Railways and Canals, representing that a communication has been received from the chief engineer of the Department of Railways and Canals, in regard of the bridge across the River St. Lawrence, near Quebec, now in course of construction, reading as follows:—

OFFICE OF THE DEPUTY MINISTER AND CHIEF ENGINEER,
OTTAWA, ONT., July 9, 1903.

L. K. JONES, Esq.,
Secretary, Department Railways and Canals,
Ottawa.

SIR,—Certain questions are at present under consideration and discussion between Mr. Theodore Cooper, the consulting engineer of the Quebec Bridge Company, and myself, involving the expediency of adopting some slight modifications of the specification for the superstructure of the bridge across the St. Lawrence river, now in course of construction by that company, attached to the subsidy contract made with them; Mr. Cooper having prepared detailed plans and specifications of such superstructure which call for special consideration.

Mr. Cooper is a bridge engineer of high standing in New York, and a man of repute and reliability. He has made a very careful study of the necessities of this superstructure, which, I may say, was especially imperative in view of the unusual magnitude of the span and of the general design of the work. His modifications may therefore reasonably be considered to be in the best interests of the work, and being engaged continuously upon the work during construction Mr. Cooper will be in the best position to note the requirements of the structure as the work progresses.

In a work of this character and magnitude it is highly important that no delay should arise from causes not absolutely unavoidable, to hinder the steady prosecution of construction, and there is reason to believe that the company require immediate instructions to proceed.

In connection with the foregoing I would suggest that the department be authorized to employ a competent bridge engineer to examine from time to time the detailed drawings of each part of the bridge as prepared, and to approve of or correct them as to him may seem necessary, submitting them for final acceptance to the chief engineer of Railways and Canals.

I have the honour to be, sir,
Your obedient servant,

COLLINGWOOD SCHREIBER,
Chief Engineer.

The minister recognizing the point urged by the chief engineer that there should be no hindrance thrown in the way of the parties engaged in the construction of the bridge superstructure, and considering that under the circumstances the course suggested by him is the best that could be adopted for the avoidance of delay, recommends that authority be given for leaving the matter in the hands of the chief engineer to the extent expressed in his communication, it to be understood that any action taken under his authority in respect of the said bridge shall be regarded and treated as in no way a violation of the company's subsidy contract dated the 12th of

November, 1900, which contract, if carried out in accordance with the decisions of the chief engineer and to his satisfaction, shall be deemed to have been properly fulfilled.

The committee submit the same for approval.

JOHN J. McGEE,
Clerk of the Privy Council.

Mr. Schreiber's principal recommendation was 'that the department be authorized to employ a competent bridge engineer to examine from time to time the detail drawings of each part of the bridge as prepared, and to approve of or correct them as to him may seem necessary, submitting them for final acceptance to the chief engineer of Railways and Canals.' In other words, it was his intention to place the final control of the bridge construction in the hands of a specially chosen bridge expert, who would be an employee of the department, and who would report directly to the deputy minister. As soon as the order in council was passed, inquiry was commenced for a suitable engineer.

The policy of Mr. Schreiber was not in accordance with the wishes of the Quebec Bridge Company and its associates—(see letters, Hoare to Cooper, July 1, 1903 (Exhibit 70 I); Parent to Fitzpatrick, June 29, 1903 (Exhibit 70 T); Fitzpatrick to Parent, July 18, 1903 (Exhibit 73 C)—and as soon as Mr. Cooper fully understood the deputy minister's plans he protested vigorously. His position is very clearly set forth in the following letter:—

NEW YORK, July 31, 1903.

DEAR MR. HOARE,—I am in receipt of papers from Mr. Schreiber which surprise me. He is to select an engineer in New York who will examine from time to time the plans, approve or correct the same as to him may seem necessary, &c.

This puts me in the position of a subordinate, which I cannot accept.

It does not relieve the situation a bit. Such an engineer must either be given liberty to do what he thinks best or he must have the very instructions which I have sought, stating to what extent there may be modifications from the general specifications, if any are to be allowed.

In either case he becomes the engineer in whom trust and confidence are reposed.

It seems to me a very simple matter for the chief engineer of Railways and Canals to decide that the 'original specifications must be rigidly carried out,' or 'that certain modifications are approved,' or 'that the company has perfect liberty to carry out the work to the best advantage, provided the efficiency of the original contract be not reduced.' I would then know where I stand.

I have written to Mr. Schreiber that I do not see how such an engineer could facilitate the progress of the work or allow me to take any responsible steps independently of his consent.

Yours truly,
THEODORE COOPER.

On July 30, 1903, Mr. Cooper wrote to Mr. Deans, advising him of Mr. Schreiber's programme, and Mr. Deans intervened actively. The following letters show very clearly that the Phœnix Bridge Company heartily supported Mr. Cooper in his contention, and that the Quebec Bridge Company was in full sympathy with their views:—

(Exhibit No. 74 W.)

July 31, 1903.

E. A. HOARE, Esq.,
Chief Engineer, Quebec Bridge Company,
Quebec, Canada.

DEAR MR. HOARE,—I was greatly exercised this morning upon receiving a letter from Mr. Cooper under date of July 30, stating that he had received from Mr.

Schreiber copy of the 'order in council,' and also a letter from Mr. Schreiber. In this letter Mr. Schreiber states he has asked for authority to employ a competent bridge engineer to examine from time to time the detail drawings of each part of the bridge as prepared and to *approve of or correct them* as to him may seem necessary, *submitting these for final acceptance* to the chief engineer of Railways and Canals. Mr. Schreiber further says, 'I have not yet named an *engineer in New York to consult with you,* but will do so without unnecessary delay, and in the meantime I think you may safely go to work on the plans.'

The seriousness of this action I have not the least doubt you will appreciate immediately. It leaves the entire matter 'up in the air,' and much worse than the condition we were all trying to avoid—which was to save most important time, and that when Cooper once approved our designs and details it would be final and accepted by the department. This is why I understand you secured the 'order in council.' It practically brings all matters to a standstill, as neither Mr. Cooper or ourselves would know where we stand until this new hand could be consulted with, and even then we would only know as each plan was passed upon.

I cannot impress upon you too strongly the necessity of taking immediate action to stop any such plan as suggested by Mr. Schreiber.

When you consider that the entire feeling and action of Mr. Cooper's was to save the Quebec Bridge Company needless expense, without the least sacrifice in the design or efficiency of the structure, it has certainly proven a thankless task for all concerned, and unless this present action upon Mr. Schreiber's part is immediately stopped the entire business will be in a worse condition than if it had been let entirely alone.

I am trying to reach you by 'phone, as I appreciate the necessity of immediate action.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

(Exhibit 70 L.)

(Letterhead of Phœnix Bridge Company.)

PHŒNIXVILLE, PA., July 31, 1903.

THEODORE COOPER, Esq.,
Consulting Engineer,
35 Broadway, New York, N.Y.

DEAR MR. COOPER,—To say that I was surprised by the contents of your letter of July 30 is putting it mildly. I am trying to reach Mr. Hoare by 'phone. In addition, I have wired him, and have also written a strong letter expressing my feeling in the matter.

The suggested action by Mr. Schreiber would place the business in a much worse condition than it was originally in. The 'order in council' was taken solely to save time and to have your approval of our details final and binding on the government—it simply being necessary to have Mr. Schreiber's signature as a matter of form. It has certainly proven to be a thankless task so far, in trying to save the Quebec Bridge Company a large amount of money, without in the least affecting the efficiency of the structure.

We of course agree with you that we are at a standstill until this matter is settled, as certainly the matter of a new engineer is an uncertain quantity at present.

I cannot but believe that a trip to Quebec by yourself and myself would tend to clear the situation.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

(Exhibit No. 70 M.)

(Letterhead of Phœnix Bridge Company.)

PHŒNIXVILLE, PA., August 1, 1903.

Mr. THEO. COOPER, C.E.,
35 Broadway, New York, N.Y.

DEAR MR. COOPER,—I talked with Mr. Hoare over the 'phone yesterday (the service was not very satisfactory), and also wired him two long messages, and have received his reply, stating that 'he will take up the question with parties at Ottawa, and that we should go ahead, and if anything turns up to cause *trouble tell Cooper to let me know at once*.' I have written him again, and urged him to stop entirely this proposed plan, and explaining that the sole purpose of the order in council was to give you the final authority to settle all details, the government approval being a mere formality, and in this way save time which was so valuable. I personally think it would have been much better to have had Douglas as originally proposed rather than to have the present plan carried out; but we must insist upon having the whole matter stopped.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

(Exhibit No. 80 P.)

(Telegram.)

August 3, 1903.

E. A. HOARE, Chief Engineer,
Quebec Bridge Company,
Quebec, Canada.

I found Cooper had written and wired you, and feels much more strongly than I do the serious result of any such action. It would be disastrous to have proposed appointment finally made. You and I should see Schreiber in Ottawa at once, and come to some better understanding. As it now stands nothing can be done on plans. Answer to Phœnixville.

JNO. STERLING DEANS.

Mr. Cooper went to Ottawa and discussed the situation with Mr. Schreiber, who, as a result of this conference, made a further recommendation to the minister under date of August 13, 1903 (Exhibit 65). This recommendation is embodied in the order in council passed on August 15, 1903 (Exhibit 18).

The text of this order in council is as follows:—

EXTRACT from a report of a committee of the Honourable the Privy Council, approved by His Excellency on the 15th August, 1903.

On a memorandum dated August 13, 1903, from the Minister of Railways and Canals, representing that by an order in council of July 21, 1903, authority was given, in accordance with a suggestion made by the chief engineer of the Department of Railways and Canals, for the employment of a competent bridge engineer to examine from time to time detail drawings of the superstructure of the bridge across the River St. Lawrence, near Quebec, now in course of construction, in view of certain modifications suggested by the consulting engineer of the Bridge Company; the said plans to be submitted for final acceptance to the chief engineer of the Department of Railways and Canals.

The minister further represents that the chief engineer has this day reported, stating that, as the result of the personal interview had with the company's consult-

ing engineer, he would advise that, provided the efficiency of the structure be fully maintained up to that defined in the original specifications attached to the company's contract, the new loadings proposed by their consulting engineer be accepted; all detail parts of the structure to be, however, as efficient for their particular function as the main members for theirs, the efficiency of all such details to be determined by the principles governing the best modern practice, and by the experience gained through actual test; all plans to be submitted to the chief engineer, and until his approval has been given not to be adopted for the work.

The minister recommends that authority be given for following the course so advised by the chief engineer, the order in council of July 21 last to be modified accordingly.

The committee submit the same for approval.

JOHN J. McGEE,
Clerk of the Privy Council.

Mr. Cooper's interpretation of the order in council of August 15 was that it gave him an absolutely free hand, provided efficiency was maintained up to the standard of the specifications attached to the subsidy contract.

Necessarily throughout the development of the design of the structure cases would arise when further modifications of the written specifications would appear desirable. Such cases did arise, and were met from time to time by Mr. Cooper. In such cases he proceeded according to his interpretation of the order in council, and did not submit further opinions to the government engineers for approval.

In this connection, Mr. Schreiber differs from Mr. Cooper, as the following extract from his evidence shows:—

Q. Considering the relation of Mr. Cooper to the Quebec Bridge and Railway Company, and your opinion of Mr. Cooper's ability, and the relation of the government with the Quebec Bridge and Railway Company, would you consider that Mr. Cooper would have the power or authority to amend the specifications for the work from time to time as he might consider necessary or desirable, and would those amendments be tacitly accepted by all parties concerned?

A. (Mr. SCHREIBER).—No, I think not. They would have to be submitted to me, and they would come before our bridge engineer—before the bridge engineer of the Department of Railways and Canals—before they would be accepted.

Q. So that, unless we can find a formal acceptance of the changes or alterations made in the specifications we would have to consider them as unauthorized?

A. (Mr. SCHREIBER).—Certainly.

There is, however, no evidence to show that Mr. Schreiber even questioned any decision made by Mr. Cooper or in any way interfered with him. We consider that in this Mr. Cooper was acting, as he believed, in the best interests of the work.

A copy of the order in council was sent to the Phœnix Bridge Company, so they were aware of its conditions, one of which was: 'all plans to be submitted to the chief engineer (Mr. Schreiber), and until his approval has been given not to be adopted for the work.' This condition also was embodied in explicit form in the contract between the Phœnix Bridge Company and the Quebec Bridge Company, and yet, the engineer of the Phœnix Bridge Company when asked, 'Did you consider the approval of the plans by the Department of Railways and Canals a condition precedent to the fabrication of the bridge,' answered, 'No.'

The specifications thus officially amended by authority of order in council were transmitted to the Phœnix Bridge Company. When asked, 'Did you fully concur in all the amendments made in the specifications, having in mind that you were endeavouring to produce the best possible bridge,' Mr. Szlapka, the designing engineer of the Phœnix Bridge Company, answered, 'The amendments made in the specifications by Mr. Cooper were not subject to my approval.'

The action of Mr. Schreiber at this time and subsequently can only be explained on the assumption that he considered the order in council of August 15, 1903, to be a direction to him to place the responsibility for the building of the bridge entirely in Mr. Cooper's hands. Mr. Cooper's amendments were according to Mr. Douglas' evidence, accepted and used by the department in subsequent examinations of plans (see Evidence), and Mr. Cooper's signature was considered by the department practically as a final warrant of the sufficiency of the plans (see Evidence).

That the proceedings of the department were irregular, and that Mr. Cooper was assuming a degree of authority not in keeeping with the wording of the order in council of August 15, 1903, was clear to the Quebec Bridge Company, as the following letter shows:—

(Exhibit No. 81 C.)

(Letterhead, Quebec Bridge and Railway Company.)

QUEBEC, May 27, 1907.

J. S. DEANS, Esq.,
Chief Engineer, Phœnix Bridge Company,
Phœnixville, Pa.

DEAR SIR,—In reply to your letter of the 24th inst., I am aware that you are doing everything that is possible to hasten the forwarding of the plans for approval by the government, except that much time might have been saved if Mr. Cooper had signed the tracings instead of having to sign so many blue prints.

The signature of the consulting engineer does not comply with the government regulations. The order in council passed some years ago only authorized certain modifications in the specification and details from time to time, if found necessary. The obligations under contracts, with the company and the government still remaining, viz., that no work is to be proceeded with or estimates paid until the final plans have been passed through the various stages required by the government in the Department of Railways and Canals. This is the point they are objecting to. Understand that it is not myself that is raising any question, but I am only endeavouring to bring you in line with the contracts. The government has passed no order in council cancelling your obligation to have all your plans approved at Ottawa before any metal is fabricated. We are under very close investigation now.

Yours truly,

E. A. HOARE.

It should be stated that the Quebec Bridge and Railway Company was throughout fully advised of what was being done at New York and Phœnixville, and did not make any objection to the authority assumed by Mr. Cooper or to the acceptance of that authority by the Phœnix Bridge Company, notwithstanding provision to the contrary existing in the contract. This letter also indicates a more active supervision on the part of the government than had previously been exercised.

The Phœnix Bridge Company was immediately advised of the terms of the order in council of August 15, 1903 (see letter, Cooper to Hoare, August 21, 1903), but being fully aware of the arguments and influences that had brought about the enactment of that order, they concluded that it was intended to grant exactly what Hon. S. N. Parent had asked for in his letter of June 29, 1903 (Exhibit 70 J).

Mr. Deans and Mr. Szlapka in their evidence (see Evidence) make it very clear that they considered Mr. Cooper's pronouncements final, and not liable to alteration either by the Quebec Bridge and Railway Company or by the Dominion Government.

In the opinion of the commission it is always desirable, when an entirely novel problem is to be solved, to have the advice of several engineers upon the unproven

features of the design before attempting to execute it. Having accepted the government's decision to depend upon the advice of only one man, the authorities thereafter acted in accordance with the best knowledge of the time; and the most competent engineers would have endorsed the concentration of responsibility upon the most experienced and able man.

In effect, after August 15, 1903, instructions given by Mr. Theodore Cooper from time to time were the specifications. In the offices of the Phœnix Bridge Company and in the works of the Phœnix Iron Company the Hoare specifications as amended by Mr. Cooper were recognized as official and were so used (see Evidence, and exhibits 99, 100, 101 and 102). It was recognized by these companies that Mr. Cooper had authority to alter any requirements of the specifications, and it is in evidence that this authority was not infrequently exercised.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY,
J. GALBRAITH.

APPENDIX No. 7.

A DESCRIPTION OF THE ORGANIZATIONS AND STAFFS MAINTAINED BY THE DIFFERENT CORPORATIONS INTERESTED IN THE ERECTION OF THE BRIDGE.

There were four parties directly interested in the building of the bridge, viz.: The Canadian Government, the Quebec Bridge and Railway Company, the Phœnix Bridge Company and the Phœnix Iron Company. Each had its own staff to take charge of the portions of the work in which it was interested.

The commissioners made the personal acquaintance of all the senior officials concerned, and discussed with each of them the duties he was called upon to perform. Evidence has been secured giving the previous experience of these men, their fitness for their several positions, and their duties.

The Dominion government was represented by the deputy minister of the Department of Railways and Canals and his assistants; two deputy ministers and three inspectors having been connected with the work.

The government's interests are set forth clearly in the Subsidy Agreement of November 12, 1900 (Exhibit 12) and in the Guarantee Act of 1903 (Exhibit 1), and throughout the work the Quebec Bridge and Railway Company recognized its obligations to the government by requiring its contractors to do their work in such a manner that it would be acceptable to the government.

Although the deputy minister of the department was charged with the duty of examining the plans and specifications, all of which were subject to his approval, checking up the monthly estimates which were the basis for payments, and exercising general oversight of the work up to the time of its final acceptance, in reality the whole responsibility for specifications, plans and construction was upon the officials of the Quebec Bridge and Railway Company, its interests being identical with those of the government, Mr. Cooper's special qualifications having been officially recognized in the orders in council of July 21 and August 15, 1903 (see Evidence).

The issue at the time previous to the passing of the order in council of August 15 referred to, was whether Mr. Cooper's approvals were to be subject to cancellation on the advice of an expert engineer employed by the department or not. By the order in council of August 15, 1903, the government practically decided that Mr. Cooper's decisions were to be final, and neither Mr. Schreiber nor his successor, Mr. Butler, at any time, interfered with his control of the technical features of the undertaking. Mr. Cooper's understanding of the situation was the same, and this indicates clearly both the government's position and that of Mr. Cooper on this question.

The Quebec Bridge and Railway Company maintained in its service and employed on the work a chief engineer, a consulting engineer, two erection inspectors and four mill and shop inspectors. The chief engineer, Mr. E. A. Hoare, M. Inst. C.E., had an extensive experience as a railway engineer, and had done most of the company's preliminary work. The record of his professional experience will be found in full in his evidence (see Evidence). Mr. Hoare had a high reputation for integrity, good judgment and devotion to duty. From the standpoints of personal character and knowledge of Quebec and its people, no better man could have been found, and the evidence throughout shows that to the best of his ability the company was faithfully served. There is, however, nothing in Mr. Hoare's record that would indicate that he had the technical knowledge to direct the work in all of its branches.

The company's directors do not seem to have realized the importance of the duties pertaining to Mr. Hoare's position and (see Parent to Holgate, January 11, 1908), while believing that he was not competent to control the work, they still gave him the position, the powers and emoluments of the office of chief engineer.

While we can only consider this as a mistake on the part of the Quebec Bridge and Railway Company, yet we regret to say that such appointments are by no means uncommon, and it must be recognized that in many cases good executive ability is valued more highly or considered of more importance than special professional knowledge.

Mr. Hoare personally considered that he was in general control of the construction, and that everything was under his jurisdiction except the approval of plans; the evidence shows that he gave much personal time to the oversight of the fabrication of the material, to inspection of the erection and the preparation of the estimates; it also shows that he lacked a comprehensive grasp of the work that was being done by the inspectors, and that although his subordinates entertained the highest personal regard for him they did not look to him for advice when technical difficulties arose.

Mr. Theodore Cooper, of New York, was the consulting engineer. In the extent of his experience and in reputation for integrity, professional judgment and acumen, Mr. Cooper had few equals on this continent, and his appointment would have been generally approved. Mr. Cooper's strict duties were to examine, correct and approve the plans prepared by the contractors, and to give engineering advice to Mr. Hoare when requested. Mr. Cooper and his chief assistant, Mr. Bernt Berger, carried on a most thorough and painstaking examination of the plans. Mr. Cooper appointed both shop and erection inspectors for reasons explained in his evidence, and had these inspectors report fully and regularly to him. Mr. Cooper states that he greatly desired to build this bridge as his final work, and he gave it careful attention. His professional standing was so high that his appointment left no further anxiety about the outcome in the minds of all most closely concerned. As the event proved, his connection with the work produced in general a false feeling of security. His approval of any plan was considered by every one to be final, and he has accepted absolute responsibility for the two great engineering changes that were made during the progress of the work—the lengthening of the main span and the changes in the specification and the adopted unit stresses. In considering Mr. Cooper's part in this undertaking, it should be remembered that he was an elderly man, rapidly approach-

ing seventy, and of such infirm health that he was only rarely permitted to leave New York.

Mr. Cooper assumed a position of great responsibility, and agreed to accept an inadequate salary for his services. No provision was made by the Quebec Bridge Company for a staff to assist him, nor is there any evidence to show that he asked for the appointment of such a staff. He endeavoured to maintain the necessary assistants out of his own salary, which was itself too small for his personal services, and he did a great deal of detail work which could have been satisfactorily done by a junior. The result of this was that he had no time to investigate the soundness of the data and theories which were being used in the designing, and consequently allowed fundamental errors to pass by him unchallenged. The detection and correction of these fundamental errors is a distinctive duty of the consulting engineer, and we are compelled to recognize that in undertaking to do his work without sufficient staff or sufficient remuneration both he and his employers are to blame, but it lay with himself to demand that these matters be remedied.

During the construction of the substructure, Mr. Cooper visited the bridge site on several occasions, but did not visit the bridge during the erection of the superstructure. He visited the Phœnix Iron Company's shops but three times during the fabrication of the structure.

During erection, Mr. Cooper, upon receipt of information from Mr. McLure, ordered certain work on the erection to be stopped for correction. This order was communicated by him to Mr. Hoare, who stopped the work accordingly.

In the sense that the inspectors looked to Mr. Cooper for advice and directions almost entirely and that he appointed them and issued instructions to them, and also that he dealt directly with the contractors, he assumed many of the duties of a chief engineer. Owing to the special nature of the work, he was the only one in the employment of the Quebec Bridge Company who was capable of assuming these duties. He was not authorized to act in this capacity, nor was he able to visit the bridge during its erection.

Norman R. McLure was an inspector assisting Mr. Edwards in the shops up to the beginning of the erection, when he acted as inspector of erection, being employed during the winter as an inspector in the shops. He was appointed by Mr. Cooper with Mr. Hoare's concurrence. He was responsible to both Mr. Cooper and Mr. Hoare, and received instructions from both, but reported to Mr. Hoare principally upon matters regarding monthly estimates, and to Mr. Cooper upon matters of construction. Mr. McLure had definite instructions in writing as to duties from Mr. Cooper (see Evidence), but had none from Mr. Hoare. Mr. McLure is a technical man, a graduate of Princeton University (1904), and previous to the Quebec bridge work was inspector of bridges for the New York, Ontario and Western Railway, and in so far as his experience fitted him, performed his duties well and is a painstaking and capable engineer. He had not full authority on the work, and depended on Mr. Cooper for all technical advice and instructions.

We are at a loss to understand why Mr. Cooper under the circumstances did not place a more experienced man in full local charge of the inspection of erection. We must recognize, however, that the power of making such an appointment did not rest with Mr. Cooper, and that Mr. Hoare has stated in evidence his conviction of his own ability to handle the work.

Mr. E. R. Kinloch acted as inspector of workmanship throughout erection, having been appointed by Mr. Hoare and was responsible to him. Mr. Kinloch's experience on bridge work as given in his evidence shows that while without technical training he had been connected with the building of several heavy structures, and was thoroughly capable of handling ordinary bridge erection. His duties were to watch the structure closely, and to see that the erection work, and particularly the riveting, were properly done and in accordance with the instructions issued by the Phœnix Bridge

Company. While the Phœnix Bridge Company did not recognize his authority, they co-operated cordially with him to the common end of endeavouring to obtain good work. Mr. McLure and Mr. Kinloch worked independently, but all Mr. Kinloch's observations and criticisms were reported to Mr. McLure, and these have added a great deal to the value of the records. Mr. Kinloch was thoroughly at home in his work, and executed his duties carefully and intelligently. He appears from the evidence to have been a keen observer and fully impressed with the importance of his duties. We are, however, convinced that the bridge was too large for one man to thoroughly cover in detail all the work that was entrusted to Mr. Kinloch, and that there should have been more inspectors of equal ability.

Mr. E. L. Edwards was chief inspector of shop work, and was appointed by Mr. Cooper with the approval of Mr. Hoare. He reported to both Mr. Cooper and Mr. Hoare. The circumstances of his appointment are stated by Mr. Cooper in his evidence, and Mr. Edwards' experience as an inspector is given in full in his own evidence. His duties were to see that the metal supplied by the rolling mills came fully up to the requirements of the specifications and that it was properly tested; he sent the test reports regularly to Mr. Hoare, and visited Mr. Cooper for instructions every month or when anything irregular happened. He had also to see that the finished members corresponded in dimensions exactly with the approved plans, and that the methods of fabrication that were used were in each case most accurate and satisfactory.

The test records and the list of shop errors detected by the inspectors are evidence as to how Mr. Edwards performed his duties.

Mr. I. W. Meeser was Mr. Edwards' assistant, and his inspection was more particularly directed to the shopwork. He had ample experience in shopwork, having been trained as a machinist, and was at one time subforeman in the shops of the Phœnix Iron Company. The commissioners satisfied themselves during their visit to Phœnixville that both Mr. Edwards and Mr. Meeser thoroughly understood the work they had undertaken. The commissioners are not, however, satisfied that the shop inspection as arranged for by the Quebec Bridge and Railway Company would have been as thorough as it was if it had not been aided throughout by the hearty co-operation of the officials of the Phœnix Bridge Company and of the Phœnix Iron Company. The staff was too small; and it is our opinion that the Quebec Bridge Company would have shown better judgment had it employed a larger staff under the direction of an independent man of wider technical knowledge and who would have been sufficiently forceful to hold his own against the contractors.

Messrs. Keenan and Ostrom acted as inspectors in the rolling mills at Harrisburg and Pittsburg, respectively. There is no evidence of any serious defect in the metal supplied by these establishments, and it may be concluded that the inspection was thorough and creditable.

As a whole the staff was inefficient and not well organized. The excellence of the work done must be largely attributed to the ambition of the constructors to do the work to the very best of their ability; the organization was weak in the absence of a fully competent engineer of erection and of a forceful chief of staff for the inspection of shopwork.

The officials of the Phœnix Bridge Company most closely connected with the Quebec bridge were the chief engineer, the designing engineer, the engineer in charge of details, the shop inspector, the superintendent of erection, the erection foreman, the resident engineer of surveys, and the resident engineer on erection.

The chief engineer was Mr. Deans, who has occupied this position for many years, and is widely and favourably known as an experienced bridge builder. Mr. Deans' personal duties are the general oversight of all work being executed by his company. He may be fairly described as its chief business manager, and as such conducted all the negotiations leading up to the Quebec bridge contracts. From the nature of his

work it is not possible for him to be closely in touch with the planning and execution of technical details; these were in the direct charge of his two principal assistants, Mr. Szlapka and Mr. Milliken, acting under his general instructions. Mr. Deans was very active in the performance of his duties, kept closely in touch with the progress of the work in all departments and generally managed the execution of the contract.

Mr. Deans' actions in the month of August, 1907, and his judgment, as shown by the correspondence and evidence, were lacking in caution, and show a failure to appreciate emergencies that arose.

The designing engineer was Mr. Szlapka, who had received a thorough technical education in Germany, and has been with the Phœnix Bridge Company for about twenty-seven years, having held his present position for twenty-one years. A list of the more important structures that have been built by this company to Mr. Szlapka's designs will be found in the evidence, and shows that previous to 1903 his ability as a designer had been thoroughly tried, and that his experience was wide. As usual in present bridge company organizations, Mr. Szlapka's work has been confined to his own department and his personal knowledge of the work of transportation and erection is limited. The evidence shows that Mr. Cooper, whose faculty of direct and unsparing criticism is well known, had every confidence in the ability of Mr. Szlapka, and on previous works had had good opportunity to form his estimate of him. Mr. Szlapka was responsible for the entire work of designing, and the commissioners are satisfied from their personal investigations at Phœnixville that this was conducted with care and energy. Mr. Szlapka's mistakes and errors, to which the disaster is directly attributed by the commissioners, are discussed elsewhere.

The engineer in charge of details was Mr. Charles Scheidl. Mr. Scheidl had received a technical education in Germany, and has been with the Phœnix Bridge Company for twenty-four years, during eighteen of which he has held his present position. His work in connection with the Quebec bridge is clearly and fully set forth in his evidence, and, briefly stated, consisted of preparing the shop drawings from the general outlines of design that had been determined by Mr. Szlapka. The accuracy with which this work was done is proved by the records of the shop inspectors and of the erectors, and was of the highest grade. Upon Mr. Scheidl was laid the burden of being personally responsible for the accuracy of every one of the shop drawings.

Mr. E. T. Morris was shop inspector for the Phœnix Bridge Company, his position being a permanent one. His duties were similar to those of Messrs. Edwards and Meeser, and his employment practically provided for an additional inspection of the work in the shop. He reported to Mr. Deans and Mr. Szlapka, and kept a record of all errors detected and of the methods adopted for their correction. An examination of the 'field corrections' reported by the resident engineer of erection will show how thoroughly this shop inspection was done, and by comparison of the records we find that the work of Mr. Morris was even more thorough and exact than that of Messrs. Edwards and Meeser. It is proper to credit the thoroughness with which this work was performed not solely to Mr. Morris but also to Messrs. W. H. Reeves, Deans and Norris, whose emphatic instructions concerning inspection he had received.

Mr. A. B. Milliken was superintendent of erection, having general jurisdiction over the handling of all the contracts of the company after the material was delivered to it by the Phœnix Iron Company. He has occupied his present position for about seventeen years. A list of the most important structures that he has erected is given in his evidence. Mr. Milliken did not confine his attention to the Quebec bridge work, but had the execution of several other contracts to look after at the same time. The evidence shows that he spent much time at the site, and was always closely in touch with work. The system of reports of progress established in his department was very thorough.

Mr. Milliken reported to Mr. Deans, and when on the work did not interfere with the jurisdiction of Mr. Yenser, who was in charge, but simply advised him. His work

throughout appears to have been thoroughly and carefully done. The system of erection was jointly designed by the engineering and erection departments of the Phœnix Bridge Company, the details of the members, their connections, the travellers and the general order of erection being determined by the engineers, and the equipment of plant and tackle by the erection department. Mr. Milliken appointed Mr. Yenser as foreman on the Quebec bridge.

Mr. B. A. Yenser was Mr. Milliken's subordinate in charge of erection, and was in absolute local authority. He had been a bridge erector for many years, and had worked for the Phœnix Bridge Company for about fifteen years. In Mr. Deans' evidence will be found a statement of the more important structures erected by Mr. Yenser, and that gentleman is described as 'having shown unusual qualities as an erector, being careful and conscientious, and having had experience in the handling of men.' It should be noted that Mr. Yenser had absolutely no authority to vary the programme of erection, which was arranged in Phœnixville and furnished to him in a book of instructions with accompanying plans. His position was largely an executive one, his duties being to carry out positive instructions and to see that the forces employed were worked to their full efficiency. He had orders to exercise extraordinary care in the inspection of the tackle and all handling appliances. Mr. Yenser had no technical training, and his position did not call for it. His action in continuing erection on August 28, 1907, was immediately referred to his engineering superiors and was approved. The evidence shows that he was an able and forceful superintendent, and that he went to his death with supreme confidence in the judgment of his superiors at Phœnixville.

Mr. A. H. Birks, the resident engineer of erection, who also perished in the disaster, had complete confidence in the ability and efficiency of the Phœnix Bridge Company's designers, whose abilities he had had ample opportunity to observe. The personality of Mr. Birks is described, and his record in the performance of his duties is stated in Mr. Deans' testimony. It will be there noted that Mr. Birks' experience was rather limited. He had received a thorough training in the design of the erection plant. His duties were to inspect the material as it arrived on the bridge for erection, to see that it was properly placed, and to watch the erectors to see that the programme of erection as laid down in the Phœnixville written instructions was minutely followed. The evidence shows that these duties were performed with intelligence and fidelity. Mr. Birks prepared all technical reports for transmission to Phœnixville, and advised Mr. Yenser on matters calling for engineering knowledge.

Mr. F. A. Cudworth was resident engineer in charge of surveys. No question of importance affecting Mr. Cudworth's work has come up in the progress of this inquiry, and it is sufficient to say that his duties were faithfully and ably attended to. The Phœnixville office depended upon him principally for reports of the movements of the various parts of the truss as the erection proceeded, and his observations are matters of record.

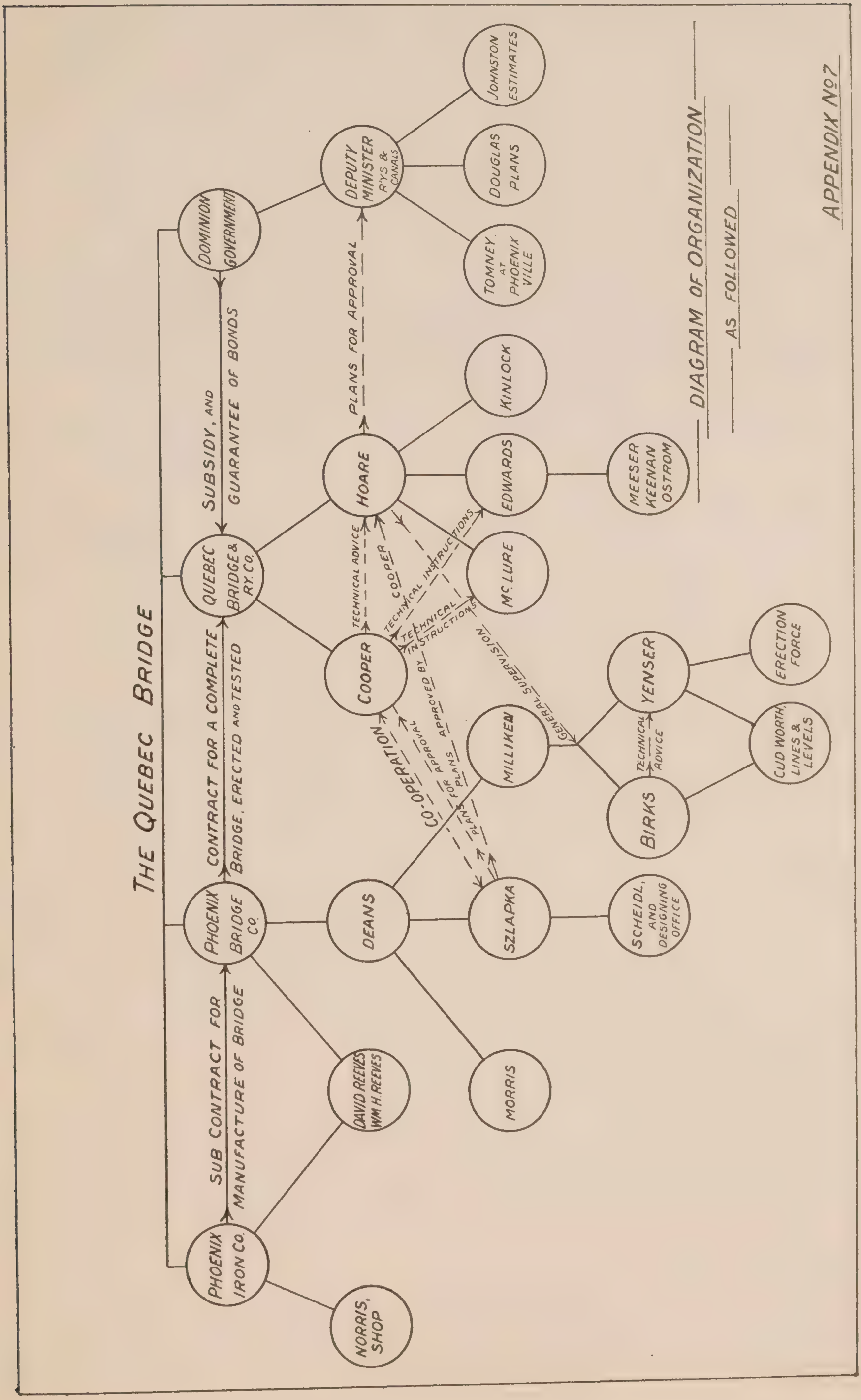
In general, it may be said that this staff was highly efficient, the men were well trained, and had ample experience in the class of work that they were called upon to do, and there is throughout evidence of great pride in their individual connection with the undertaking and of determination to do their utmost to make it a success in every way. The commissioners are of opinion, however, that the Phœnix Bridge Company erred in judgment and showed a failure to appreciate the magnitude and difficulties of the work that it had undertaken when it did not provide as part of this organization an engineer of erection who, by virtue of technical training and long experience on large bridge work, was fitted to take complete local control of the erection. In this they followed usual practice, which, however, was not applicable to this particular work.

The manager for the Phœnix Iron Company was Mr. Norris, who has been prominently connected with the company since 1898, and who became manager in

in 1900. Under his management the works of the company have been altered and enlarged and the output materially increased. Mr. Norris' endeavour to secure thoroughly good material and good workmanship for the Quebec bridge is set forth at length in his testimony, and his conduct of the work throughout is, in the opinion of the commissioners, commendable for its carefulness, thoroughness and energy.

The commissioners are of opinion that the works of the Phœnix Iron Company are efficiently managed and operated.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY,
J. GALBRAITH.


THE QUEBEC BRIDGE
DOMINION GOVERNMENT
SUBSIDY, AND GUARANTEE OF BONDS
QUEBEC BRIDGE & RY. CO.
CONTRACT FOR A COMPLETE BRIDGE, ERECTED AND TESTED
PHOENIX BRIDGE CO.
SUB CONTRACT FOR MANUFACTURE OF BRIDGE
PHOENIX IRON CO.
NORRIS, SHOP
DAVID REEVES WM H. REEVES
DEPUTY MINISTER R'YS & CANALS
JOHNSTON ESTIMATES
DOUGLAS PLANS
TOMNEY AT PHOENIX VILLE
PLANS FOR APPROVAL
HOARE
KINLOCK
EDWARDS
MEESER KEENAN OSTROM
McLURE
TECHNICAL ADVICE
COOPER
TECHNICAL INSTRUCTIONS
TECHNICAL INSTRUCTIONS
COOPER
CO-OPERATION
FOR APPROVAL
APPROVED BY
PLANS FOR PLANS
GENERAL SUPERVISION
MILLIKEN
YENSER
ERECTION FORCE
CUDWORTH, LINES & LEVELS
TECHNICAL ADVICE
BIRKS
DEANS
SZLAPKA
SCHEIDL, AND DESIGNING OFFICE
MORRIS
DIAGRAM OF ORGANIZATION
AS FOLLOWED
APPENDIX No 7

APPENDIX No. 8.

A HISTORY OF THE DEVELOPMENT OF THE PLANS AND OF THE METHODS FOLLOWED IN THE DESIGNING OFFICE.

The first preliminary plan of the Quebec bridge was made by the Phœnix Bridge Company for the Quebec Bridge Company, and is dated November 30, 1897 (Exhibit 94). A second plan was made December 7, 1897 (Exhibit 95), showing the lower chord curved. In response to our inquiry Mr. Szlapka states that the change from the straight to the curved lower chord was made for the sake of artistic appearance, either form being considered by him structurally satisfactory.

There are three other plans dated February 17, 1899, two of which show the lower chord of the anchor arm arched at both ends, and the other shows the anchor arm arched only at the main pier. In general outline this last plan was almost identical with that of the final design.

All of these five general preliminary plans are drawn for a river span of 1,600 feet. The plan of November 30, 1897, shows the cross-section of the river correctly, which indicates that information of this nature had been received from the Quebec Bridge Company prior to that date.

The plan made by the Phœnix Bridge Company and dated December 7, 1897 (Exhibit 95), is identical as to bridge outline with the plan dated January 13, 1898, and filed in the Department of Railways and Canals by the Quebec Bridge Company (Exhibit 3).

The plan which accompanied the tender of March 31, 1899 (Exhibit 96), was one of the three plans dated February 17, 1899. This design and others of the same date, some of which were competitive designs, are shown on drawing No. 33.

Two plans were made by the Phœnix Bridge Company, both dated April 22, 1900. Both of these show the anchor arm with a complete arch in the lower chord, but the river span is 1,723 feet, and in the other 1,800 feet. These plans were made subsequent to the awarding of the contract for the bridge on April 12, 1900, but before Mr. Cooper had advised the adoption of the 1,800-foot span. Another general plan was made dated May 6, 1900, showing the bridge generally as it was intended to be built. A further plan was made by the Phœnix Bridge Company, dated October 6, 1900, similar to the last mentioned plan, but with the title of the 'Quebec Bridge Company,' and on April 14, 1901, a further and last preliminary general plan was made by the Phœnix Bridge Company, which is practically the same as the former plan and bears the same outline as the constructed bridge. All these preliminary plans were made by the Phœnix Bridge Company.

On April 12, 1900, the contract embracing the anchorage steel work was signed. The plans for this work were developed in the regular course, and the work was done accordingly. In this agreement the Phœnix Bridge Company were awarded the contract for all the steelwork of the whole structure, and agreed to proceed with detail plans. On December 19, 1900, the contract for the two approach spans was signed. The plans for this work were developed in the regular course, and the work was finished in due course.

While the approach spans were simple truss spans of usual design for such structures, and complete in themselves, the anchorages for the cantilever bridge involved calculations of the main structure, in order that the uplift could be determined. Such calculations as were necessary to ascertain this were made on assumed

data. The work was ordered June 15, 1900, and forthwith designed and constructed with 'a liberal allowance for increase of uplift,' as the exact uplift could not be ascertained, the weight of the structure itself not being then known and only approximately estimated.

A general study of the details of the bridge was made by Mr. Scheidl during the months of January, February and March, 1902. This study involved the consideration of the outlines of the bridge and of the general stress sheets which had been prepared. The method of connecting the suspended span to the cantilever arms, the details of the shoes for the main posts, and the details of the anchorages were considered; the detailing of the panel points and intersections of the suspended span were worked out; then followed the arrangement of the top chord packing for the cantilever and anchor arms, the panel points and intersections, the main posts and pedestals. These studies not being considered as other than tentative, the weights were not computed at that time as a basis for new stress sheets. The real preliminary work intended for final results was begun in July, 1903, after the receipt of the revised specifications, the contract having been signed provisionally on June 19.

The following is an outline of these preliminary studies. First, the determination of the normal lengths of all the bridge members; then studies of all plate and trussed floor beams and stringers; of transverse bracings, details of main shoes, pedestals, connecting chords and bracing of same.

The packing of the eye bars was then taken up, then the details for anchorages and the transfer of wind stresses and anchor piers. Then followed the detail of the anchor arm panel points, commencing with the end lower chords; then the web intersections. Similar studies were made for the cantilever arms and the suspended span.

When the details for the anchor arm were completed and those for the cantilever arm partially completed, the weights of all details were calculated and final anchor arm stress sheets computed. This was the beginning of the shop-drawing period; only the anchor towers had then been shop detailed.

The dead load concentration upon which the final make-up of the members of the anchor arm was based were as follows:—

Half suspended span	4,842,000	lbs.
Cantilever arm	13,205,200	"
Anchor arm	13,317,600	"

The corresponding concentrations as determined on June 25, 1907, were found to be:—

Half suspended span	5,694,000	lbs.
Cantilever arm	15,804,000	"
Anchor arm	17,318,000	"

(See also drawing No. 3.) The total steel in these concentrations was about 35,316,000 lbs.

The difference between these two sets of concentrations indicate a fundamental error in the calculations for the bridge. In a properly computed bridge the assumed dead load concentrations upon which the make-up of the members is based should agree closely with the weight computed from the dimensions in the finished design and with the actual weights (see clause 3 of the specifications, which provides: '3. The dead weight used for calculating stresses must not be less than the actual weight of structure when completed').

The error consists not so much in the assumed concentrations being incorrect, for that is more or less unavoidable, but in the fact that a recomputation of weights based upon the cross-sections already determined and with sufficient allowance for doubtful details was not made, and the process of approximation continued until satisfactory agreement was reached. In bridges of ordinary design and dimensions

experience has furnished data sufficient to enable the designer to estimate the weight so accurately that a recomputation is not always necessary. In this case, however, the unique character of the design, the magnitude of the span and the high unit stresses specified demanded that no risks should be taken by failing to adopt this method of approximation and checking.

In computing the make-up of the members of the cantilever arm the same dead load calculations were assumed for the cantilever arm and for the suspended span as in the computations for the anchor arm, and the above comment will apply with equal force to the design of the cantilever arm.

The failure to make the necessary recomputations can be attributed in part to the pressure of work in the designing offices and to the confidence of Mr. Szlapka in the correctness of his assumed dead load concentrations. Mr. Cooper shared this confidence, as he approved the stress sheets.

The dates of approval of the various stress sheets by Mr. Cooper are as follows:—

Suspended span, March 29, 1904.
Anchor arm, June 30, 1904.
Cantilever arm, May 25, 1905.

Mr. Cooper's examination with regard to the question will be found in the evidence. Mr. Cooper says:—

'In computing the dead load strains I was furnished by Mr. Szlapka with a diagram, dated May 12, 1904, which gave the dead load concentrations for the anchor and cantilever arms, Quebec bridge.

'These dead load concentrations vary at every point. I asked Mr. Szlapka when this was presented to me whether it was carefully and properly estimated. He states that he had his best men to carefully estimate the weight at each point, and that this was a correct arrangement of the final weight to the best of his belief. As I had no other means of determining these weights, the plans not being yet submitted to me, I assumed them to be correct, and used them in determining my strains. I did, however, check these weights in the following manner: I added together all the concentrated loadings, deducted the allowance for floor and timber which he states here especially, and found that the resultant weight was abundant to cover the assumed estimated weight of the structure.'

Early in 1905 the drawings of the anchor arm were practically complete, and it was possible to compute the weight of the anchor arm within say two per cent of the actual weight. There is no evidence to show that this was done by the Phœnix Bridge Company or by Mr. Cooper. Had such a computation been made at that time, when but a small portion of the work had passed through the shop, and erection had not begun, the serious error of the assumed dead load would have been immediately detected.

Shopwork began in July, 1904, and the record shows that at the end of December, 1904, eight panels of anchor arm lower chords had been completed ready for shipment. The demands of the shop on the drawing office no doubt contributed to prevent a recomputation of the stress sheets and the early discovery of the error.

Mr. Cooper did not become aware of the error until he received Mr. Edwards' report on material of February 1, 1906. At this time, the anchor arm, tower and two panels of the cantilever arm were fabricated, and six panels of the anchor arm were in place. Realizing that there was no remedy, and believing that the increased stresses were still within the limit of safety, Mr. Cooper permitted the work to proceed. He estimated that the increase in unit strains due to this error was from seven to ten per cent.

No progressive record of computed weights was made and kept in the designing offices for the purpose of checking the estimated concentrations used in the stress sheets, as will be seen from the following correspondence with Mr. Deans:—

MONTREAL, January 25, 1908.

Messrs. the Phœnix Bridge Company,
Phœnixville, Pa.

GENTLEMEN,—I have been requested to ascertain from you the process of system you used in the drawing office, whereby the weights were estimated and ascertained, and what record of estimated and actual weights of parts was kept in the drawing office. Was it your practice as soon as a drawing was completed to make an estimate of the weight of that part, and was this work done systematically so as to be of service in checking the original calculations of the bridge, and were the weights estimated from the drawing of service to you in furnishing data for design ? If you can give me a list of these actual weights, with the dates at which they were ascertained, either from the estimate made from the drawing or from actual weight, whichever was first, it would, I think, give the information desired.

I would be obliged to you for an early reply.

Truly yours,

HENRY HOLGATE.

PHŒNIXVILLE, PA., January 31, 1908.

HENRY HOLGATE, Esq.,
Chairman, Royal Commission,
Montreal, Canada.

DEAR SIR,—Replying to your letter January 25.

When the shop drawings of the heaviest and largest pieces were partially finished, sketches were prepared showing their approximate weights and extreme dimensions. These sketches were sent to the transportation companies to secure routing and method of loading. No weights were figured of any pieces of ordinary dimensions, where no difficulty whatever was expected in loading. After the shop drawings of the most important members were finished and approved by the consulting engineer, then their weights were figured with care for comparison with the shipping weights. No other record was kept in the drawing room outside of these itemized weights on forging lists of the estimated and actual shipping weights.

Our shipping invoices give the actual weights, marks and dates of shipment of all pieces in the bridge. We have no extra copy of these, but no doubt you could get the loan of Mr. Hoare's record.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

Before commencing the shop drawings a full understanding was arrived at between the drawing office and the erection department regarding the positions of field splices and other details which might affect the erection. The large traveller was designed after consultation between the two departments.

Before shop drawings could be made for the larger pieces, arrangements had to be completed with the transportation companies. This involved the making of transportation drawings for the purpose of avoiding all difficulties which might arise during the transportation with regard to rolling stock, curves and bridges.

The exact dimensions of the various members had to be determined, so that under normal loading the normal configuration should obtain. This involved the computation of all the alterations of lengths and of position in members from the first position of the anchor arm lower chords on the false works to the final configuration of the bridge when complete and sustaining its normal load.

The methods of erection were fully considered in planning the details of the shop drawings. All the work, including the preliminary detailing already described, was under the charge of one assistant engineer, Mr. Chas. Scheidl. Each shop drawing, when completed, was fully checked. Copies were then forwarded to the consulting engineer for approval. When approved copies were returned to the Phœnix Bridge Company, which sent copies to the chief engineer of the Quebec Bridge Company to be forwarded to the Department of Railways and Canals. The department returned one approved copy to the Phœnix Bridge Company, and in accordance with the terms of the contract, the receipt of the plans approved by the Department of Railways and Canals was the authority for the Phœnix Bridge Company to construct the work.

All shop drawings were executed in the best style of draughtsmanship, and gave all necessary information for the shop and to some extent for the erection.

The most careful methods of checking were employed. At no time during the progress of the office work were more than eighteen men employed. The rate of progress depended upon the rate at which Mr. Scheidl could perform his work, and would not have been hastened by the employment of a greater number of draughtsmen. (For fuller details see the evidence of Mr. Szlapka and Mr. Scheidl.)

The north and south halves of the bridge being identical, the members for each were constructed from the same drawings simultaneously.

The annexed table shows that the drawings were sent to the shops as soon as the approval of Mr. Cooper was obtained, and that the approval of the Department of Railways and Canals, while necessary, was regarded as being purely formal.

LOWER CHORDS OF SOUTH ANCHOR ARM.

SHOWING dates at which various operations on these were performed.

Drawing Number.	Location	Sent to Mr. Cooper for Approval.	Returned by Mr. Cooper Approved.	Sent to Mr. E. A. Hoare	Approved by Dept. Rys. and Canals.	Drawings Received in Shops.	Templetes Completed.	Starting of Punching.	Finished Punching.	Completed Ready for Shipment.
		1904.	1904.	1904.	1904.	1904.	1904.	1904.	1904.	1904.
1	End panel.....	6 July........	10 July........	....	21 October.....	22 July........	1 September...	5 August......	28 September...	1-19 October. 1-20 October.
2	Second panel...	16 August......	20 August......	29 August......	21 October.....	25 August......	25 September...	19 September...	1 October.....	1-27 October. 1-24 October.
3	Third panel....	10 September...	14 September...	22 September...	1 November...	16 September...	5 October.....	11 October.....	22 October.....	1- 8 November. 1- 3 November.
4	Fourth panel...	19 September...	24 September...	28 September...	1 November...	26 September...	11 October.....	20 October.....	28 October.....	1-12 November. 1-14 November.
5	Fifth panel.....	16 September...	24 September...	8 October.....	1 November...	29 September...	31 October.....	2 November...	17 November...	1-25 November. 1-26 November.
6	Sixth panel....	14 September...	24 September...	13 October.....	1 November...	4 October.....	1 November...	10 November...	17 November...	1- 3 December. 1- 6 December.
7	Seventh panel..	7 October.....	13 October.....	21 October.....	12 November...	17 October.....	14 November...	21 November...	3 December...	1-13 December. 1-17 December.
8	Eighth panel...	14 October.....	18 October. ...	4 November...	23 November...	24 October.....	18 November...	4 December...	17 December...	1-24 December. 1-31 December.
9	Ninth panel....	19 October.....	23 October.....	4 November...	23 November...	28 October.....	25 November...	13 December...	31 December...	1905. 1-12 January. 1-16 January.
10	Tenth panel....	21 October.....	25 October.....	23 November...	19 December...	7 November...	28 November...	28 December...	1905. 5 January.....	1-18 January. 1-19 January.
	Column........	1	2	3	4	5	6	7	8	9

Compare column 4 with 5. Compare column 4 with 9.

The following is a condensed diary of the work more or less connected with the drawing office in connection with the structure as erected:—

April 12, 1900.—Contract for anchorages signed.

December 19, 1900.—Contract for two approach spans signed.

January, February, March, 1902.—Tentative studies of details for main structure.

July, 1903.—Preliminary studies for final design of main structure began on receipt of the revised specifications. Contract signed provisionally on June 19.

July 23, 1903.—Studies of floor system began. Mr. Szlapka decides to arrange his work so as to complete the shop drawings for the anchor and cantilever arms not later than August 31, 1904, giving the shops eight months to complete twenty million pounds, so that erection might begin May 1, 1903.

January to May, 1904.—Computation of stress sheets and make-up of anchor arm.

March to December, 1904.—Computation of stress sheets and make-up of cantilever arm.

February 19, 1904.—General drawing and stress sheets of suspended span sent to Mr. Cooper.

March 21, 1904.—Mr. Deans instructed Mr. Szlapka to push all work with the utmost despatch.

March 29, 1904.—Stress sheets of suspended span approved by Mr. Cooper.

April 8, 1904.—Mr. Szlapka advises Mr. Hoare that weight of bridge would not be more than five per cent above the estimate, or, say, 62,720,000 pounds.

April, 1904.—Large traveller designed, and weight determined for computing erection stresses.

May, 1904.—General detail drawing suspended span approved by Mr. Cooper.

May 3, 1904.—Details of anchor bents approved by Mr. Cooper.

May 13, 1904.—Mr. Szlapka sends Mr. Cooper dead load concentrations for cantilever and anchor arm, so that Mr. Cooper might check his stress sheets.

May 23, 1904.—Preliminary study of shoes and pedestals sent to Mr. Cooper. Also complete calculations for anchor arm. Also first shop drawing for anchor bent.

May, 1904.—All typical drawings of top and bottom panel points approved by Mr. Cooper.

June 2, 1904.—Complete stress sheet for anchor arm taken to Mr. Cooper by Mr. Szlapka.

June 6, 1904.—Revised plan of anchor eyebars sent to Mr. Cooper.

June 30, 1904.—Mr. Cooper approves anchor arm stress sheet.

July, 1904.—Plate floor beams and stringers approved by Mr. Cooper.

July 10, 1904.—First lower chord plans approved by Mr. Cooper, and work begun on them in shop.

July 11, 1904.—Copies of anchor arm stress sheet sent to Mr. Hoare for transmission to Department of Railways and Canals.

July 28, 1904.—Top chords approved by Mr. Cooper. After this drawings completed and forwarded to Mr. Cooper in a continuous stream.

August, 1904.—Shop drawings of two end panels approved by Mr. Cooper.

The following letter from Mr. Deans to Mr. Hoare describes the situation as at October 8, 1904:—

October 8, 1904.

E. A. Hoare, Esq.,
Chief Engineer, Quebec Bridge and Railway Company,
Quebec, Canada.

Dear Sir,—We find we have not received from the government engineer the approval of any main chord sections. As explained to you some time ago, we have been working at great disadvantage to ourselves in being compelled to confine our

office work to the anchor arm, in order that everything might be done that it is possible to do to be ready early next spring to start the erection of the anchor arm. There was too much work to do in the time allotted after the financial arrangements were made and work ordered ahead. We have not therefore been able to complete our stress sheets for the cantilever arm and for the suspended span, it being necessary to await the completion of all details, not only of the permanent structure, but also the details and rigging of the main traveller, that we may know exactly the total weight coming at each panel point.

We have as you know, sent to the Canadian engineers, through your office, the stress sheets for the anchor arm, covering the chords which have not been approved, and we would kindly ask that they be examined and prints sent to us with their approval as soon as possible. The engineers have everything that is necessary to check these chords, although we thoroughly appreciate they would like to have before them these stress sheets of the entire bridge, and these will be sent with the least possible delay.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

November 19, 1904.—Plan of centre post approved.

January, 1905.—Series of eyebar tests made.

March 3, 1905.—Drawings for main shoes sent to shops.

May 25, 1905.—Mr. Cooper approves stress sheet for cantilever arm.

July 12, 1905.—First detail drawing cantilever arm chord No. 9 sent to Mr. Cooper.

July 13, 1905.—Stress sheet for cantilever arm approved by Department of Railways and Canals. Anchor arm at this stage nearly all fabricated. Mr. Szlapka expected to finish the shop drawings for first two panels of cantilever arm by September 1, and all the drawings for the bridge by March 15, 1907.

July 20, 1905.—Mr. Cooper and Mr. Szlapka discuss the testing of riveted links and other matters. Use of slightly higher steel for eyebars and some corrections in camber. All satisfactorily agreed upon.

August 11, 1905.—First lower chord sections of anchor arm erected in position, and practically whole of anchor arm fabricated, a large amount having been delivered at bridge site.

June 14, 1906.—Development of drawings so far advanced that the Phœnix Bridge Company made a closer estimate of the weight of the steel in the structure, which, including the anchorages, was placed at 73,000,500 pounds. The weight finally was estimated at 73,312,504 pounds. The actual weight averaged about one per cent heavier than the weight computed from the drawings. (See attached statement of weights.)

November 26, 1906.—The south anchor arm and nearly all the south cantilever arm erected.

February 1, 1907.—Stress sheets of the suspended span revised.

March 15, 1907.—The last drawing completed, being that of the lower chord of centre panel of suspended span.

June 25, 1907 to October 8, 1907.—Dead load concentrations for suspended span, cantilever arm and anchor arm revised and new cross sectional areas for members of bridge computed for purposes of comparison with actual cross-sections.

HENRY HOLGATE,
Chairman.

J. G. G. KERRY,

J. GALBRAITH.

QUEBEC BRIDGE—SOUTH HALF.

STATEMENT SHOWING COMPARISON OF ACTUAL WEIGHTS AND WEIGHTS FIGURED FROM COMPLETED DRAWINGS.

Order No.	Description.	Figured Weight.	Total.	Actual Weight.	Total.
		Lbs.	Lbs.	Lbs.	Lbs.
602	Anchorage eyebars and pins	219,829		223,100	
604	Anchor shells and bracing	374,697		371,843	
	Anchorage	....	594,526	....	594,943
606	Anchor arm trusses	8,085,621		8,142,803	
608	Anchor arm eyebars	3,188,361		3,209,014	
610	Anchor arm pins	229,058		229,255	
	Anchor arm truss system	....	11,503,040	....	11,581,072
613	Anchor arm floor beams and stringers	1,507,140		1,517,036	
618	Anchor arm trusses floor beams	260,832		261,510	
	Anchor arm floor system	....	1,767,972	....	1,778,546
612	Centre posts and bracing	2,676,863		2,708,560	
614	Shoes and pedestals	808,810		814,349	
	Centre post system	....	3,485,673	....	3,522,909
621	Cantilever arm trusses	8,602,086		8,724,598	
623	Cantilever arm eyebars	3,467,005		3,468,253	
625	Cantilever arm pins	330,220		329,584	
	Cantilever arm truss system	....	12,399,311	....	12,522,435
627	Cantilever arm floor beams and stringers	1,732,290		1,770,892	
629	Cantilever arm trussed floor beams	290,435		296,206	
	Cantilever arm floor system	....	2,022,725	....	2,067,098
631	Suspended span trusses	3,307,590		3,379,293	
633	Suspended span eyebars	342,340		343,280	
635	Suspended span pins	35,710		35,460	
	Suspended span truss system	....	3,685,640	....	3,758,033
637	Suspended span floor beams and stringers	1,197,365	1,197,365	1,214,905	1,214,905
	For one-half of the bridge	36,656,252		37,039,941	
	The whole bridge	....	73,312,504	....	74,079,882

Actual weight in excess of weight estimated from drawings, 767,378 pounds.
Percentage of errors, 1.03 per cent.
Actual weight is 101.05 per cent of figured weight.
Figured weight is 98.95 per cent of actual weight.

SEPTEMBER 25th, 1907.

APPENDIX No. 9.

MATERIAL, SHOPWORK AND INSPECTION.

The steel supplied for the bridge was made to meet the requirements of the Hoare specifications, with the exception that Mr. Cooper, finding that the tests on the full-sized eyebars were running a little low, called for the use of a slightly higher material for eyebar blanks.

The Hoare specifications called for an ordinary grade of structural steel very similar to the regular output of the mills. The testing requirements were not onerous but were in accordance with current practice. Some reference to this will be made in Appendix No. 18.

The behaviour of the metal, as evidenced by the wreck, was so good that the commission was convinced that the disaster could not be traced to the furnaces or rolling mills. Its examination of these was accordingly rather general in character.

The following amounts of metal were supplied by the different mills:—

Phœnix Iron Company, shapes..	16,575,888 lbs.
Central Iron and Steel Company, eyebar blanks..	14,827,400 "
Central Iron and Steel Company, plates...	27,240,100 "
Carnegie Steel Company, plates...	13,822,000 "
Bethlehem Steel Company, pins..	993,600 "

The commission visited the works of the Phœnix Iron Company, of the Central Iron and Steel Company, and of the Pennsylvania Steel Company, the latter corporation having supplied a large tonnage of slabs to the rolling mills of the Central Iron and Steel Company.

On our inspections we were accompanied by the mill inspectors employed by the Quebec Bridge and Railway Company, and the details of the manufacture of the steel, of the rolling of the shapes and plates and of the work of the inspection were explained fully to us both by these gentlemen and by the superintendents in charge of the various works.

We desire to acknowledge here the courtesies extended to us by Mr. J. B. Bailey, manager of the Central Iron and Steel Company, and by Mr. Reynolds, vice-president of the Pennsylvania Steel Company.

The tests of material called for in the Hoare specifications were regularly made by the rolling mills under the supervision of the inspectors for the Quebec Bridge and Railway Company, and the reports of these tests are filed as Exhibit No. 28. An examination of these records, there being in the neighbourhood of five thousand tests in all, shows that there was nothting abnormal about any of the material, and that it satisfactorily met the requirements of the specifications.

Full-sized tests of some seventy eyebars were also made in the large machine at Phœnixville in accordance with the requirements of the specifications. The results of these tests are given in Exhibits Nos. 28 and 86, and it will be noted that a number of the bars tested did not quite come up to specifications. The results of these tests were referred to Mr. Cooper, who agreed to accept a certain number of weak bars, but raised the rolling mill specification so that there would be no further difficulty of this nature. These full-sized tests were made on finished eyebars, prepared in all respects as were the eyebars that were used in the bridge.

Mr. Cooper's statements (Cooper to Hoare, August 4, 1903) that 'the various members of this bridge will exceed anything heretofore made, and will tax to the utmost the manufacturing appliances of the time,' is a fair description of the work that the Phœnix Iron Company had undertaken to perform.

When the Phœnix Bridge Company provisionally signed the final contract of June 19, 1903, its subcontractor, the Phœnix Iron Company, was not fully equipped for the carrying out of the work, and additions and changes had to be made both to its buildings and to its plant.

The study that had been given to improvement of equipment preparatory to the acceptance of the Quebec contract is set forth in the evidence of Mr. Norris; and the Phœnix Iron Company was ready to commence making the necessary changes as soon as the contract was accepted.

The total expenditure then made on improvements was over $220,000, divided as follows:—

Enlargement and improvement of eyebar plant..	$ 40,000
Alterations of buildings and installation of overhead cranes sufficiently powerful to handle weights of 100 tons..	110,000

New machinery, including a 64″ double rotary planer for facing the compression chords, a plate straightener for large thick plates, hydraulic shears for heavy sections, larger boring mills, large vertical planer, and sundry alterations to other machinery. 70,000

This expenditure was necessary before the Quebec work could be properly handled, and is an evidence of careful preparation for that undertaking. The additions themselves constitute a permanent improvement to the Iron Company's plant, and are now in constant use as part of its regular working equipment.

The evidence shows that Mr. Reeves and Mr. Norris fully appreciated the difficulty of manufacturing the large and complicated pieces of the Quebec bridge, and that the various superintendents and foremen were warned to give more than usual attention to the execution of the work. As a preliminary, a full-sized wooden model of one of the panel points of the lower chord of the anchor arm was made, and remained set up for the inspection of the shopmen. All details, such as the heads of rivets, &c., were shown on this model, so that the shopmen could realize the mechanical accuracy that was necessary in order that the several members meeting at a point would go together in the field.

The commissioners spent some days in the workshops with the Iron Company's foremen and with the inspectors for the Quebec Bridge and Railway Company in order to familiarize themselves with the work of fabrication and inspection. There was nothing peculiar to the Quebec work other than the great size and weight of the pieces to be handled, and the usual bridge shop methods were followed, the provisions of Mr. Cooper's standard specifications having to be observed for workmanship.

It was the obvious intention of the Iron Company to do a first-class piece of work, and it is in evidence that the management impressed not only on its own officials but also upon the employees of the Quebec Bridge and Railway Company its desircials but also upon the employees of the Quebec Bridge and Railway Company its desire that the shop inspection should be thorough and rigorous.

All pieces were inspected twice, once by the regular shop inspector employed by the Phœnix Bridge Company and again by the inspectors for the Quebec Bridge and Railway Company. In the more delicate work the inspectors had orders not only to test the finished pieces, but also to test the setting in the machines before the final cuts were made.

It was the practice of the shop to make the duplicate pieces for the north and south halves of the bridge at the same time; so that the bridge material now lying in Belair yard was manufactured under exactly the same conditions as that which was erected from the south shore. The commission spent some time examining the material in Belair yard, for the purpose of satisfying itself concerning the finish of the workmanship on the lower chords. This was found to be by no means perfect, but the errors measured were of small amount. The shops were defective in that they lacked a well founded floor for the assembling of the heavy pieces. The methods adopted were also defective in that adjoining compression members were not fitted together before shipment. Some of the minor, but by no means negligible errors discovered in Belair yard would have been detected by this fitting, and it is a customary practice on heavy work. That errors similar to those observed at Belair existed on the south half of the bridge there can be no doubt, and Mr. Kinloch (see evidence) states that such errors were observed by him. That these minor errors at the joints contributed in some degree to the final disaster is probable, but our criticism in this case is not of the shopwork, which was of a fair grade. The fault lies in a design which called for an accuracy beyond the working limits of good shop practice.

The errors now being discussed are differences of length of the several ribs making up one chord and irregularities of surface at the field joints of the lower chords. The chord faces are found to be slightly dished and not true. It is not

possible to determine by analysis the result of these slight errors, the larger of which would not exceed 3/64 of an inch in dimensions.

The inspectors were instructed to work to the nearest one-sixtyfourth of an inch; but such accuracy is hardly practicable. We do not consider that such high accuracy can be maintained, at least not without fitting together adjoining pieces in the shop.

It is probable that some portion of the errors noted at Belair was due to the unavoidable racking of the members while in transport.

Both sets of inspectors worked with steel tapes that had been carefully compared, and kept books of record stating the errors discovered in their inspections and the methods adopted in correcting them. In cases of difficulty the question was referred to Mr. Szlapka for instructions, and occasionally to Mr. Cooper.

Some few errors (see Exhibit 91, 'Field corrections') escaped detection until the work was being erected in the field; the drafting room and not the machine shop was responsible for several of these. None of these final errors were of a serious nature, and the necessary corrections were made without difficulty.

Mr. Edwards has recorded the following number of errors:—

In the anchor arms.—Twenty-three in the stringers, 2 in the floor beams, 17 in the lower chords, 20 in the main posts, 7 in the hangers, 4 in the eyebars, 6 in the pedestals and shoes, 1 in the main diagonals, 14 in the laterals, 15 in the struts, 2 in the pins, 8 in the plates and in knee braces, giving in all 119 errors.

In the cantilever arms and suspended span.—Twenty-seven in the lower chords, 10 in the floor beams, 8 in the stringers, 8 in the diagonals, 4 in the struts, 4 in the hangers, 34 in the main posts, 4 in the laterals and 5 in the eyebars, giving in all 104 errors.

As the inspection requirements were more severe than is customary on ordinary bridge work, and as the shopmen had never been called upon to handle work of such magnitude before, it was natural that a number of errors should be made, and that this number should decrease proportionately as the conditions of the work became better known to the men.

It will be noted from the figures given above that such a decrease in the number of shop errors did take place, and in the correspondence the better quality of the workmanship on the cantilever arms, when compared with that on the anchor arms, is referred to from time to time.

Mr. Kinloch states in his evidence that in spite of the magnitude and difficulty of the work, which would reasonably account for an unusual number of shop errors, the number actually found during erection was not in excess of what would be regularly expected on much simpler work.

Mr. Edwards' list of errors, which is not so ample as that prepared by Mr. Morris, looks, in a statement, to be rather serious, but when the number and magnitude of the pieces are remembered it cannot be considered to indicate carelessness or insufficiency in the shops. Some errors will always occur.

On the whole we consider that the inspection of the material and the work both in the mills and shops was reasonably efficient, and that the collapse of the bridge is not attributable to want of care in either.

Some special shopwork errors that occasioned a good deal of correspondence are referred to elsewhere.

The evidence shows that Mr. Cooper was seriously annoyed at the number of shop errors reported and reprimanded the inspectors very sharply, but the ease with which the structure was erected indicates that their work was fairly well done.

The lines of the several ribs in the chords are known to have been wavy to the extent of from ½-inch to ¾-inch (see evidence), but errors of this size and kind do not appear to have been considered a cause of anxiety. The existence of these wavy bends had been noticed by the shop inspectors, and had been reported both to Mr. Szlapka and to Mr. Cooper.

We find no evidence to show that the seriousness of such minor errors in the compression chords and posts was appreciated by the engineers or was ever impressed by them upon the inspectors. The necessity of detail accuracy in compression members is referred to in Appendix No. 16.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY,
J. GALBRAITH,

APPENDIX No. 10.

TRANSPORTATION AND ERECTION.

In the practice of steel bridge design, details are of vital importance; and connections which may appear to be simple and satisfactory frequently prove to be impossible of execution. The complete study of the details therefore involves patient and skilful work, and necessarily occupies a great deal of time.

A large portion of the time spent on the Quebec bridge plans by the designers was devoted to the study of practical details.

Four main principles had to be observed:

(1) The size of the metal shapes and plates called for in the bills of material had to be limited to the dimensions that the rolling mills could furnish. It will be noted by reference to Appendix No. 9 that a large tonnage of metal was made for this bridge by the Carnegie Steel Company, neither the Phœnix Iron Company nor the Central Iron and Steel Company being able to make the larger plates.

(2) The members had to be designed so that the machines in the shop could make them. It will be noted by reference to Appendix No. 9 that the Phœnix Iron Company had to provide a number of new machines with which to manufacture the Quebec bridge. These machines were not novel in design; they were simply larger than those previously used by the Phœnix Iron Company, and were required on account of the greater size of the parts entering into the work.

(3) The members had to be designed of such size and weight that the railways could transport them. To ensure this it was necessary to know and comply with the clearances and weight limits of several different railroads. For some of the members special cars were provided, so equipped as to make safe transportation a reasonable certainty. It may be noted that one member of the north half of the bridge has been lying in the Phœnixville yard for about three years awaiting the renewal of certain railroad bridges over which it would have to pass to reach Belair yard.

(4) The members had to be designed so that they could be easily and quickly erected to place with the appliances provided. This made it necessary for the designers to thoroughly study the system and appliances for erection. The erection equipment provided was almost entirely new, and much of it was built specially for this bridge.

The capacity of the erection equipment was sufficient, although demands made upon it were very great. Some of the members handled weighed 100 tons, and one lift of two panels of eyebars was 145 tons. This was lifted and placed in position in the upper chord of the bridge without difficulty, proving the capacity and perfection of the apparatus used in erection.

It should be said that the errors and mistakes of the Phœnix Bridge Company in connection with the bridge were made in the design, and that its work in detailing, shopwork and erection was excellent. The care and forethought given to the execu-

tion of the work cannot be better described than it is in the evidence of Messrs. Deans and Scheidl. We therefore add only a few explanatory remarks to those statements.

Some of the photographs (Exhibits 126 and 127) show the size and complexity of parts of the bridge.

The bridge members were loaded on cars by the Phœnix Iron Company, and were shipped either to the Chaudière or to the Belair storage yards, which are indicated on drawing No. 1 (map). The equipment at these yards for handling the members is described by Mr. Deans and illustrated by the photographs.

The facilities for loading, unloading and transporting the material were entirely satisfactory so far as the safe delivery of the members is concerned.

But four cases of accidents during transportation from the shop to the bridge are reported.

Mr. Milliken (see evidence) has given the particulars of an injury to one of the steel shells that stood on the anchor pier. This injury was due to an accident on the railway.

The accident to chord 9L anchor arm which occurred in the Chaudière storage yard, and which is frequently referred to in the evidence, is discussed in Appendix No. 11.

An accident to centre post 6R which occurred in the Chaudière yard is also referred to in Appendix No. 11. An injury occurred to one of the north side lower chords, which fell in the Phœnix Iron Company's yard, striking a centre post cap. These pieces were repaired before they were shipped, and have not yet been erected.

The work was delayed owing to lack of railway connections to the bridge site. The Quebec Bridge and Railway Company's railway line giving connection with the Chaudière storage yard was not opened for traffic until July 9, 1905, the first metal for the main spans being placed on the south anchor pier on July 22, 1905. Owing to lack of this connection, all the metal for the anchorages and approach spans, and all the material for the falseworks and traveller, had to be sent to Lévis or Quebec and taken to the bridge site on barges. The beginning of the erection of the main spans was delayed, and considerable difficulty was experienced by the contractor, owing to the congestion of the yards at Phœnixville and Belair. At the present date there are no railway connections with the bridge on the north side of the river; similar conditions existed on the south shore of the river early in 1905.

It was the duty of the Quebec Bridge and Railway Company to see that these rail connections were provided.

The erection traveller is described by Mr. Deans, and is shown in the photographs in Exhibits 126 and 127. Great attention was given to the design and equipment of this traveller, and it performed its work in a manner entirely satisfactory to the erectors. In evidence the erection workmen stated that they had never worked on a bridge on which better appliances were provided, or on which the erection programme had been more perfectly arranged. In order to hasten the erection of the bridge, which had been delayed by lack of rail connections, it was decided in January, 1906, to erect the suspended span with a small traveller, so that the big traveller might be removed to the north shore at an earlier date. This programme, which was followed, was found quite satisfactory, and it tended to increase the safety of the structure during erection, as erection stresses were thereby reduced.

At the time of the collapse of the bridge the small traveller was doing all the work of erection and the big traveller was being dismantled.

In the design of the bridge a normal configuration and loading was assumed in which the stresses in all the members were intended to be axial. In other words, under these conditions no bending stresses would exist at the various joints; under any other loading, therefore, angular changes would either take place or tend to take place at the joints: that is to say, bending stresses would exist.

The shop lengths of all members being computed so that in the normal configuration the members had the normal lengths, it resulted that during erection, when the members were under little or no stress, the whole configuration was distorted, as compared with the normal configuration. The false works upon which the anchor arm was built had therefore to be arranged to conform to the initial configuration. After the anchor arm was erected, the building of the cantilever arm gradually introduced and increased the stresses in the various members of the anchor arm, and at a certain stage the anchor arm became free from the camber blocks on the falsework, which were lowered to assist this movement.

In the original distorted form all field butt joints were in contact only at one edge, since in the normal form they would be in full contact. With the increasing loads on the cantilever arm, due to the progress of erection, these joints gradually approached the condition of full contact, and in doing so revolved about the edges in contact; in the meantime the splices were secured by bolts which could be changed as the movement at the joints improved the matching of the holes. The instructions issued by the Phœnix Bridge Company were that when the joints finally closed the splices should be permanently riveted.

It must be apparent that during the movement in question the stresses at these joints were applied first only at the edges in contact, and that it was not until the joints were fully closed that there was any possibility of uniform distribution of stress. Indeed this condition was not possible until the bridge would be completed and carrying its normal load, and the attainment of this condition would even then be dependent on the accuracy of the mechanical work at the joints.

Drawings No. 8 and 11 in this appendix show in an exaggerated manner the members in the initial distorted configuration; and drawing No. 12 shows, among other things, the records kept of the above described camber movements.

These movements were regularly and carefully observed by the Phœnix Bridge Company's engineer in charge of survey work, and Mr. Deans states that these observations agreed closely with the expected movements as calculated by the designing engineers.

The adopted scheme of erection was carefully worked out in all details before the work of erection began. The results of this study were embodied in a book of field instructions (Exhibit 60), copies of which were furnished to the principal foremen on the work and to the representatives of the Quebec Bridge Company. These instructions were imperative, and were not departed from or varied without approval of the Phœnix Bridge Company at Phœnixville.

Mr. Kinloch in his evidence, referring to these instructions said: 'In fact you had only to follow instructions and the thing would get there itself if you followed the lines laid down.' This statement coming from a bridge erector of Mr. Kinloch's experience is a tribute to the completeness of the prearranged system of erection.

There can be no doubt that the camber problem in the Quebec bridge was much more difficult than in ordinary structures on account of the magnitude of the bridge and the great size of its members.

The progress of erection is illustrated by the dated photographs and the date at which each member was erected as shown on drawing No. 6.

The actual work of erection of the bridge began July 22, 1905, and continued for that season until November 24. This work comprised six panels of the south anchor arm.

In 1906 erection was commenced April 16, and continued until November 29. At the end of this season's work the condition of all joints, as reported by Mr. Birks and Mr. Yenser, complied with the requirements of the Phœnix Bridge Company's

instructions to their employees (Exhibit 60). At this date the anchor arm and practically all of the cantilever arm were erected.

Work of erection was resumed May 1, 1907, and continued until August 29, the date of the collapse of the structure. At that date the fourth panel of the suspended span was in course of erection.

APPENDIX No. 11.

A DISCUSSION OF THE DIFFICULTIES THAT AROSE DURING ERECTION AND OF THE EVENTS AT THE TIME OF THE COLLAPSE OF THE STRUCTURE.

The contract for the construction of the main spans was made conditionally on June 19, 1903, and finally accepted by the Phœnix Bridge Company on March 15, 1904. By the 1st of August, 1904, the assembling of materials for the falseworks on the south shore had commenced, and by the beginning of September, 1904, the erection of the falsework was well under way. The wooden falsework for the supply tracks and the steel falsework for the traveller and bridge trusses were erected simultaneously, not quite one-half of the falsework being put up before December 1, 1904. The erection of the big traveller was commenced, and the storage yard at Chaudière was in working order before the end of the season of 1904.

SEASON OF 1905.

A considerable amount of material was delivered at the Chaudière yard during the winter, but the work was not pushed in the spring of 1905 because there was no rail connection between the bridge site and the Chaudière yard. This connection was completed on July 9, 1905, at which time the framework of the big traveller was being completed, and the falsework had been erected to the main pier but was not finished.

The equipment of the traveller was installed and the erection of the steelwork was commenced at the anchor pier on July 22, 1905. By the middle of September the lower chords of the anchor arm had been erected, the pedestals and feet of the centre posts were being placed and the erection of the web members and upper chords had commenced.

By the end of the season, six panels of the anchor arm, out of a total of ten, were in place. The weight of metal erected during each month is given in the monthly estimate of the chief engineer (Exhibit 42), the total amount erected during 1905 being about 10,500,000 pounds.

The work during the season proceeded satisfactorily both to the Phœnix Bridge Company and the Quebec Bridge Company. There were some difficulties which are described in the evidence. The more important of these were as follows:—

Field corrections, 1905.—The 'field' filed notices of 21 corrections and alterations with the 'office' of the Phœnix Bridge Company's erecticon department. These files up to August 29, 1907, all concern minor alterations that would facilitate erection, but do not call for comment.

Chord A 9L.—In April, 1905, this chord had a severe fall while being handled in the Chaudière yard. One of the hooks that were being used in raising it broke, and the whole chord fell, one end striking on a yard plate lying on the ground, and the other on a pile of eyebars. The drop was five feet at one end and about three feet at

the other. The chord struck in such a way that any resulting bend would have been at right angles to the deflections measured on August 27, 1907. The two lower flange angles were broken. This chord was repaired in July, 1905, in accordance with drawings received from Phœnixville, and to the satisfaction of the Quebec Bridge Company. We have examined these repairs since the fall of the bridge, and we find nothing to justify us in connecting them with the disaster. Whether the chord was strained by its fall so that it afterwards bent more readily under stress is a matter of conjecture that cannot be settled. A discussion of the failure of chord A 9L under less than its working load will be found in Appendix No. 16.

Painting.—There was some discussion because the designs were such that water and snow could lodge in many pockets of the steelwork, and that other parts of it were inaccessible for future painting. Mr. Hoare considered that this was an 'oversight' on the part of both the Phœnix Bridge Company and Mr. Cooper, and on Mr. Kinloch's advice insisted on its being remedied. No changes were made, but better provision for painting was arranged for in the members not yet built.

Masonry.—It was found necessary to delay the placing of the pedestals until the surface of the masonry upon which they were to rest was dressed level. Mr. Cooper would not permit the use of a lead plate under the pedestal, and had pieces of duck, heavily coated with red lead, used instead.

Main shoe right truss.—On placing this in position it was found that the bottom did not bear evenly on the pedestals, there being an opening parallel to the bridge centre line about 4 feet wide and perhaps $\frac{3}{16}$-inch high at the maximum. It was decided that this would close as the weight on the shoe increased, but this closing had only partially occurred up to August 29, 1907. The shop inspector (Mr. McLure in this case) states that no warp existed in the finished pieces in the shop, and that it must have been caused by handling and transportation. The matter does not call for further comment.

Lower chords—bends.—It was noticed by Mr. Kinloch that lower chords A 1R, A 2-R, A 3-R, after they were set, and before any stress came on them, did not look straight, but were wavy to the extent of perhaps $\frac{1}{2}$-inch. He discussed this matter with Messrs. Birks and McLure, and it was decided that it was of no importance. It was also noted early in September, 1905, that the openings at the lower chord splices did not correspond exactly with the erection diagrams (Exhibit 60), 'but seemed to average up about the same,' and also that the inside ribs of chords at splices 1 and 2 did not line up well.*

SEASON OF 1906.

In 1906, erection commenced on April 16, and the south anchor arm was all in place, with the exception of some decorative details, by June 27. Erection continued on south cantilever arm and this was completed, with the exception of some connecting pieces between it and the suspended span before work closed down for the year, on November 26. The total weight of metal erected during this season was about 21,000,000 pounds. Work on the north shore commenced about the middle of July, and a small portion of the falsework was in position by the end of the season.

During this season few difficulties occurred, and these were of a kind usually met with in all large work. The following quotation from Mr. McLure's report to Mr. Cooper, under date of July 21, 1906, gives a fair idea of the conditions existing on the work:—'The whole policy of the Phœnix erection department seems to be to make things safe and take no chances, which is a very satisfactory one to us, and in pursuance of this everything is being bolted up in full in cantilever arm, with the

*On drawing No. 11 the erection markings of the various members are shown, the letters R and L being used to denote the trusses on the Quebec and Montreal sides of the bridge respectively.

largest size bolts the holes will take, post and chord splices, main and subdiagonal splices, as well as all lateral and transverse bracing connections.'

Field corrections, 1906.—Fifty corrections and alterations were reported by the 'field' during this season, none of them being of a serious character as far as the safety of the bridge was concerned.

Painting.—The field inspectors for the Quebec Bridge and Railway Company recorded many minor defects both in the arrangements for future painting and with regard to the shop painting that had been done. There are few bridges built upon which this difficulty does not arise.

Centre post.—Section No. 6 of this post in the Quebec truss (C P, 6R) was injured while being handled in the Chaudière yard in April, the outstanding leg of one of the flange angles of an inner rib being broken through the slipping of a hoisting chain. This break was repaired during the summer in accordance with plans drawn by the Phœnix Bridge Company and to the satisfaction of the Quebec Bridge Company's inspectors. There is no evidence to show that this break was a cause of the collapse of the bridge. On June 2, Mr. McLure reported to Mr. Cooper that the bearing surfaces at the top of C P 1, both R and L, were not even and would not give a good bearing to the centre post aps, these surfaces being made up of the tops of the posts themselves and of two brackets attached to each. Mr. Cooper immediately wired Mr. Hoare as follows: 'Do not allow posts C P, 1, erected until top is made level. Notify McLure.' Mr. Hoare immediately issued instructions to this effect. The Phœnix Bridge Company sent Mr. Scheidl to check Mr. McLure's measurements, and the defect was finally made good in accordance with Mr. Cooper's detail instructions to Mr. McLure. The fault lay both in the fitting of the brackets and in the facing of the posts by the planer. Mr. Cooper considered such workmanship to be disgraceful; but the defects as stated to him were rather exaggerated owing to the methods of measurement adopted by the inspectors.

Compression members.—On July 20, Mr. McLure wrote to Mr. Edwards as follows:—'On a number of the compression members that we have erected—particularly on three or four anchor arm bottom chord sections, in chord 621 8-L (south cantilever arm, bottom chord), and in main diagonal sections for both anchor and cantilever arms (T 5 and T 50), and on 621 S P-5 sections (south cantilever arm sub-posts), especially the latter—in sighting from end to end, the webs in places are decidedly crooked, and show up in wavy lines apparently held that way by the lacing angles. This makes a very bad appearance, for a person seeing a member like that, and knowing it to be in compression, would at once infer that it had been overstrained sufficiently to bulge the webs. As to its actual effect in the number of cases I have figured out there is no possibility of this causing trouble, as long as the lacing in the members in question is intact.' On September 22, Mr. McLure reported to Mr. Cooper a deflection of $\frac{1}{2}$-inch in a distance of 36 feet and of $\frac{1}{4}$-inch in a distance of 17 feet in the upper section of post 3-L, cantilever arm (621 U P 3-L). Mr. Cooper replied that he did not like the distortions, but did not see that anything could be done at that stage. No effort was made to correct any of these irregularities, all of which were due either to shop errors or to racking in transportation. We do not connect these undoubted faults immediately with the disaster.

Removal of steel falsework.—In August, 1906, the Phœnix Bridge Company issued instructions covering the removal of the steel falsework bents, under members T O and P I, anchor arm. The draft of the instructions showed that the Phœnix Bridge Company expected the portions of the anchor arm near the main pier to lift first, as the weight erected on the cantilever arm increased, but desired, for convenience of erection on the north shore, to take down the bents near the anchor pier as soon as possible. On September 15, Mr. McLure reported these instructions in detail to Mr. Cooper, and asked him for directions concerning the matter; he also

reported that no lifting was yet visible at any point in the anchor arm. On September 17, Mr. Cooper directed Mr. McLure to permit the removal of the falsework, provided he was satisfied that the remaining bents would not be overloaded. On September 29, Mr. McLure reported that E P, R, had lifted clear of the falsework, and on the same day it was noticed that T O O O-R was free. After discussion in the 'field,' the blocking under T 5-Z, both R and L, was lowered $\frac{5}{8}$-inch, T 5 Z-R then swinging free. On October 2, Mr. Cooper advised Mr. McLure that he thought the intermediate bents were too high, and that he should examine them for evidences of extra loading and have them slacked down. 'The whole must be rather a matter of careful observation and judgment rather than any reference to theoretical lines.' Mr. Cooper read this letter to Mr. Szlapka, and during the following week the blocking under T 5 Z, P-4 and T O O O O was lowered on orders from Phœnixville. As this was done without notice to Mr. McLure, who had received Mr. Cooper's instructions about the falsework, he immediately protested against this failure to recognize the inspectors of the Quebec Bridge Company. A short and rather sharp controversy arose over this, which was closed on October 20, by a personal letter from Mr. Hoare to Mr. Deans, below quoted, in which Mr. Hoare very definitely asserts the importance of Mr. McLure's position as the representative of Mr. Cooper and himself, and makes it clear that no important steps are to be taken in the future without Mr. McLure's knowledge:—

(Letterhead, the Commissioners of the Trans-Continental Railway.)

QUEBEC, October 20, 1906.

DEAR DEANS,—I wish to send you a few personal lines on the following matter. Mr. McLure showed me a letter dated October 5, written by him to Mr. Milliken, respecting the relieving of steel falsework bents under anchor arm without giving him notice of such a procedure in order that Mr. Cooper first and then myself be previously notified. Mr. McLure has specific instructions to notify Mr. Cooper of any important procedure, and receive in return any instructions that may be necessary. I fancy changes were made from Phœnixville to relieve the falsework. Mr. McLure—representing the Bridge Company's officers not daily on the work—should have been immediately informed, notwithstanding the fact that you considered your instructions perfectly correct and safe. If Mr. McLure had been informed in time he could have wired Mr. Cooper your intentions without any delay to the work. I entirely endorse his letter to Mr. Milliken and to you on the subject of yours of the 8th inst. to Mr. Milliken.

Both you and Mr. Milliken appear to have misunderstood Mr. McLure's letter. He did not for a moment intend interference with erection orders from your office, but makes a plain request to be informed of important moves of the above nature, and not be ignored, in order that he may perform his duty and carry out his instructions. I regret your remarks on his lack of experience, as it was uncalled for, and as a reflection on the Bridge Company's supervision, and instead of helping matters the tendency will be to ignore general inspection orders which can be considered as given by me personally. Mr. McLure communicates daily with me and weekly with Mr. Cooper to receive instructions when necessary. I am writing you a personal and friendly letter, which I hope will receive your usual generous consideration by seeing that Mr. McLure is better informed in future by your chief representative on the work of any proceedings of importance or of the nature referred to.

Yours truly,

E. A. HOARE.

In the week ending October 29, T 5 Z, P 4, T O O O, and E P, were reported as free from the falsework, and in the following week the blocking at T O O P 2, T O O and P 3 (drawing No. 5) was lowered, P 1 swinging clear while this was being done.

By November 3, only T O O and P 2 were still bearing, and by further lowering of the blocking the whole truss was set free before November 28. This record shows clearly that the right truss rose more quickly than the left truss, and that the centre of the anchor arm remained resting upon the falsework for the longest time. In his evidence Mr. Cooper has expressed the opinion that the blocking near the centre was left too high, and that it acted as a fulcrum, permitting E P and T O to lift from the falsework at an early date, whereas, theoretically, they should have been the last to lift. On page 842 of the evidence he suggests that this condition may have produced an undue and unprovided-for strain on the anchor arm splices. There is no evidence that any serious action of this nature took place, Mr. McLure having been unable to observe any signs of stress at the suspected points, and no deformations in a vertical plane being anywhere on record. In our opinion the failure of the Phœnix Bridge Company to more closely adjust the blocking of the truss to its movements was an error of judgment, as the stresses produced by the gradual working of the truss are not calculable, and the movement should be made as free as possible.

The commission has been unable to satisfactorily determine the respective duties of Mr. Hoare and Mr. Cooper, their real positions being perhaps better brought out by the events of 1906 than by any other evidence. According to Mr. Deans (letter, Deans to Parent, April 14, 1900, Exhibit 75 K), Mr. Cooper had to approve all plans, but all other authority was vested in Mr. Hoare, and this opinion Mr. Deans continued to hold throughout the work (see evidence). According to Mr. Parent (letter, Parent to Holgate, Evidence), Mr. Hoare was practically an executive officer acting in all technical matters on the direction of Mr. Cooper, who was *de facto,* chief engineer, Mr. Cooper himself has stated that the erection plans were not subject to his authority (see evidence), and has disclaimed any responsible connection with the inspection either in the shop or in the field (see Evidence). With few exceptions, all his directions are advisory and not imperative, and he seems to have endeavoured throughout to avoid encroaching upon the privileges and rights properly pertaining to Mr. Hoare's position. He gave frequent directions to both Mr. McLure and Mr. Edwards on technical matters, but throughout the construction period (August, 1905, to August, 1907) he had practically no correspondence with Mr. Hoare. Mr. Cooper's opinions, when given, were accepted by the inspectors as instructions. The impression left with us is that throughout the work Mr. Cooper was in the position of a man forced in the interests of the work to take responsibility which did not fully belong to his position, and which he was not authorized to take, and that he avoided the assumption of authority whenever possible.

Such an organization cannot from an executive standpoint be considered entirely satisfactory. Mr. Yenser closed the season of 1906 with the following report:—

New Liverpool, P.Q., November 30, 1906.

The Phœnix Bridge Company,
Phœnixville, Pa.

Gentlemen:—

SOUTH SIDE.

I beg to report to-day that all the bolting is fully completed on all metal erected in accordance with your instructions.

The work for closing down for the winter is nearing completion. The traveller has been unrigged, and all tools are properly stored. The engines on the traveller are housed, and the shelters are now being covered with tar paper.

The storage yard is closed, and the locomotive put away.

The large scow has been beached, and preparations for putting the small scow in winter quarters are under way.

A general report will be sent you at the entire closing down for the season.

Yours truly,

B. A. YENSER.

SEASON OF 1907.

Work for the season of 1907 began in March, it being necessary to have a yard prepared to receive material on the north shore by early spring. The yard was located at Belair, close to the junction of the Canadian Pacific and National Transcontinental Railways. Work on the trusses began on May 1, but until May 31 was confined mainly to riveting. Using the big traveller, the connecting links between the cantilever arm and the suspended arm were put in, and the small traveller was built. On July 13, the erection of the suspended span was commenced, the small traveller being used, and the dismantling and removal of the big traveller was begun. Both of these operations were in progress when the bridge fell, on Thursday, August 29.

On the north shore work continued at a leisurely rate from about May 15 until the day of the accident. The north shore falsework was not fully erected by that date, there being no reason to hurry, because rail connection could not be obtained.

During this season less than 3,000,000 pounds of metal was erected. The last progress estimate (August, 1907) showed that about 34,400,000 pounds in all had been erected.

Riveting.—It had been intended to delay much of the riveting of the structure until the erection of the south half of the bridge was completed, and all joints had their full stress; but at a meeting between Mr. Cooper and Mr. Szlapka, on May 10, it was decided that riveting could be done at once at all joints where the connecting pieces had taken their full bearing. The estimate of the amount of field riveting in the south half of the bridge was as follows:—

Part of bridge.	No. of rivets.
Anchor arm and centre posts	121,000
Cantilever arm	98,700
South half of suspended span	53,300
Total	273,000

Some minor riveting was done in 1905, and in 1906 the joints of the floor beams and those near the anchor pier were riveted, but the bulk of the riveting was not started until 1907. Drawing No. 7 shows the dates on which the joints of the main trusses were riveted. The following table shows the number of rivets driven during the periods specified:—

Period.	No. of rivets driven.
During 1905	7,807
" 1906	46,301
" May, 1907	31,517
" June, 1907	26,512
" July, 1907	38,917
" August, 1907 (not including August 29)	28,019
Total	179,073

On August 3, Mr. McLure reported that the anchor arm was ninety per cent riveted, although the bottom lateral braces in panels 6, 9 and 10 were not riveted; and that forty per cent of the riveting on the cantilever arm was done. At the same date the lower chord splices at 5-6, 9-10 and 10-11 were the only chord splices in the anchor arm remaining unriveted. Throughout the season the work proceeded satisfactorily; there were practically no difficulties until after August 1.

Fourteen corrections and alterations were reported by the 'field' to the office of the erection department.

The surveys in May showed that the truss had stood up very well throughout the winter, the movement of the centre post being trifling, indicating that the stresses then existing were well within the strength of the members. On July 20, a wooden derrick that was being used in the dismantling of the big traveller was struck by lightning. The derrick mast was shattered, but no other damage was done.

The difficulties with the lower chords that finally resulted in the collapse of the bridge were noted early in the season, but those first observed were considered to be of minor importance. The joints between lower chords 5 and 6, anchor arm, remained open $\frac{1}{16}$-inch on the lower side long after all the others had closed. They finally closed shortly before the disaster, and on August 29 were being riveted. No explanation has been offered of the slow closing of these joints, and from their nearness to the falsework bents at T O O and P 2 it is possible that the pressure of the falsework may have had something to do with this.

On June 15, Mr. McLure reported to Mr. Cooper as follows:—'In riveting the bottom chord splices of south anchor arm, we have had some trouble on account of the faced ends of the two middle ribs not matching as per following sketch (the sketch shows that at the lower sides the middle ribs of the abutting chords were out of line by $\frac{1}{8}$ to $\frac{1}{4}$ inch, this offset decreasing to nothing near the mid depth of the ribs). This has occurred in four instances so far, and by using two 75-ton jacks we have been able to partly straighten out these splices, but not altogether. These were probably in this condition when erected, but owing to the presence of the bottom cover plate, it was then impossible to detect them, and it was only when this plate was removed for riveting that the inequality was noticed. The chords found in this shape were between 3 and 4, 7 and 8 and 8 and 9, in east truss, and 8 and 9 in west truss. You will note that this occurs only on inside ribs, which are provided with but a single thin splice plate each. I think that a heavy plate on each side of these ribs, bolted up tight when chords were erected, would have remedied this, i.e., drawn the ribs together till the "faced ends matched."' Mr. Cooper replied on June 17, saying:—'Make as good work of it as you can. It is not serious. It would be well to draw attention to as much care as possible in future work to get the best results in matching all the members before the full strains are brought upon them.'

It should be noted that of the four joints mentioned, those between chords 3 and 4 and 7 and 8 had originally been opened at the lower side and had come together by 'camber' movement; but the 8 and 9 joints had been set with the lower edges abutting. During the first stages of erection, the upper edges of all the ribs at a joint were exposed to view, as the upper cover plate was not in place. Mr. Kinloch, to whose practical knowledge of bridge work and powers of observation much of the excellence of Mr. McLure's report is due, states in his evidence that he observed gaps between abutting ribs as great as $\frac{3}{32}$-inch due to irregular finish of the planed ends of the chords. In the examination of the material in Belair yard the commissioners found irregularities of workmanship which would account for the conditions described above, and in our judgment these could have been avoided only by matching the chords together in the shop previous to shipment. The small gaps between abutting ends of chords closed as the pressure on the chords increased, with no result other than producing irregularity of stress, but the lateral deviations had to be corrected by the use of jacks.

As Mr. Cooper, in his evidence (see evidence), has expressed the opinion that these lower chord joints were, during erection, the weakest and the most hazardous part of the structure, and that they suffered from lack of appreciation of the necessary care to be given them, it is advisable to closely review all evidence concerning them. The chords consisted of four deep and narrow ribs latticed together and finished with square ends so that the pressure might be transmitted from one chord to the next by contact of the abutting ends. Under the system of erection adopted it was possible to place the adjoining chord ends in contact only at either the upper

or lower edges, and it was expected that the chords would gradually turn during the settlement of the bridge until the end surfaces came fully in contact, as is more fully described in Appendix No. 10. This expectation was realized. The adjoining chords were held together by eight spliced plates, an upper and a lower horizontal plate, two vertical plates on each outside rib and one vertical plate on each inner rib. The order of erection required that the lower plate should be put in position before the next chord was set; the vertical plates were next placed, and the erection of the joint was finished by bolting on the upper plate. Owing to the erection angle at the joint it was possible to use full size bolts on only one horizontal plate and on one edge, either upper or lower, of each vertical plate. The instructions with regard to the bolting were very definite, and read as follows (see Exhibit 60):—'all bottom chords to have two-thirds of all holes of web splices filled with 1-inch bolts on the outer ribs, and ⅞-inch bolts on the inner ribs, or their equivalent in smaller bolts or drifts. For top splice plate apply rule (1), (this requires that every hole shall be filled with a bolt), and never take off splice plate again, not even while driving rivets in web splices. Bottom splice plate to be bolted with bolts (two-thirds value). While driving rivets in web splices of chords, remove bottom splice plate and bolt across flanges temporary angles to keep flanges in place.' Owing to the camber openings at the joints it was found necessary in some cases to use ⅝-inch bolts, as no larger bolts could enter the holes in their erection condition.

The evidence shows that these instructions were carried out, but not with a full appreciation of their importance. Mr. Birks, who was admitted by all witnesses to have been an exceptionally accurate and painstaking inspector, examined all the bolting towards the end of the season of 1906, this examination being made on direction of Mr. Deans, and at the express request of Mr. Reeves, the president of the Phœnix Bridge Company. He reported as follows:—

All bottom chord splices in anchor arm—top plate full—bottom plate and webs 67 per cent—all joints bolted as per instructions;' and also, 'all chords in the first five panels of the cantilever arm top plate full—rest 67 per cent.' Mr. McLure's report about bolting has already been quoted, and Mr. Kinloch, in his evidence, states that the Phœnix Bridge Company's instructions about bolting were fully obeyed, but that he personally did not pay much attention to the bolting of the bottom cover plate, as he knew that it had to come off during riveting. Beauvais, the riveter, in his evidence casts some doubts upon the inspectors' reports, and we are of the opinion that the top and bottom cover plates and the splice plates for the outside ribs, all of which could be readily seen by the inspectors, were correctly bolted, but there may have been some cases of insufficient bolting on the inside ribs. Such cases were we think rare. It was intended that, as the camber openings closed, the smaller bolts should be taken out and replaced by larger bolts on all outside plates, the inner plates being difficult of access until the bottom cover plate was removed. This idea does not seem to have been followed in practice to any extent, nor is there any evidence to show that the bolting was systematically tightened up, as it worked loose with the adjustment of the structure. The evidence also shows that the bottom cover plates were left off during the whole period of riveting a joint (usually from ten days to two weeks), and that in the case of 7-8 L cantilever arm this plate was off for nearly the whole month of August, 1907. We must therefore conclude that the splice plates at the joints were rather loosely attached, and that the importance of rigidity at these points were strangely overlooked.

It should be noted that this system of bolted splices was a necessity due to the method of erection adopted, but that there was no reason why the end details of the chords and the splice plates themselves should not have been much more strongly and rigidly designed. The erection problem was unique in magnitude, particularly in the camber requirements, and the method followed by the Phœnix Bridge Company closely corresponds to that in general and successful use on smaller structures. It

is open to criticism on theoretical grounds, and it is possible that other engineers might, by other design, serve the same ends; the problem in its dimensions is so entirely new, that there is room for much study and invention in erection methods for great structures.

We know of no reason why the method adopted cannot be successfully used, but the evidence shows that the Phœnix Bridge Company failed to appreciate the important influence that end details and splices had on the strength of the chords. Steps were not taken to ensure that the work was so handled that the maximum rigidity consistent with design was secured at these joints. Considering the circumstances, we know of no good reason why the riveting should not have been much further advanced before the great stresses created by the erection of the suspended span were thrown upon the joints. The report of Mr. McLure on November 10, 1906, shows that all but eight of the forty lower chord joints were then closed and ready for riveting. Mr. Cooper has clearly stated that he did not consider that the erection methods were subject to his control, although the evidence shows that he was frequently consulted about them, both by Mr. Szlapka and by Mr. McLure. The erection problem in this case was of great importance, and the Quebec Bridge Company did not place their interests under the direct and responsible control of an experienced engineer acting solely on its behalf.

Difficulties developed almost as soon as the erection of the suspended span got well under way. On August 6, Mr. McLure reports as follows:—

NEW LIVERPOOL, P.Q., August 6, 1907.

Mr. THEODORE COOPER,
Consulting Engineer,
45 Broadway, New York City.

DEAR SIR,—In riveting up the splice between chords 8 and 7 in the west truss of south cantilever arm we found the condition of the inside ribs at splice as indicated in the following sketch (drawing No. 30).

Owing to the limited space between the two inside ribs, it would be impossible to jack this splice back, and as the condition is not nearly as bad at the top of the splice, we have proposed putting a diaphragm between the two inside ribs to cover the first five rivets up from the bottom on each side of the splice, as indicated in red in the sketch above. The splice plates being riveted on the two inside ribs, it will be necessary to cut out and redrive twenty rivets to do this. This provision, together with the top and bottom cover plates, should be sufficient to hold this splice against the thrust due to its being out of line, which thrust when under its maximum compressive stress I estimate at not over 60,000 pounds.

The Phœnixville office is being notified of this plan, and if they will approve will wire us. If this also meets with your approval, or if you wish to suggest another way to remedy the difficulty, will you please wire me at St. Romuald, P.Q., care Phœnix Bridge Company, as the riveting gangs are ready to finish riveting this splice.

Very truly yours,

N. R. McLURE.

Upon receipt of this letter, Mr. Cooper wired the Phœnix Company as follows, August 8:—

NEW YORK, August 8, 1907.

PHŒNIX BRIDGE COMPANY,
Phœnixville, Pa.

Method proposed by Quebec for splicing joints at lower 7 and 8 chords is not satisfactory. How did bend occur in both chords?

THEODORE COOPER.

And wrote Mr. McLure on August 9 as follows:—

New York, August 9, 1907.

N. R. McLure, Esq.,
Inspector for Erection, Quebec Bridge,
New Liverpool, P.Q.

Dear Sir,—Yours of the 6th regarding bent condition of lower 7 and 8 chord joint came yesterday. I wired Phœnix that the proposed method as sketched by you for repairing was not satisfactory. Also asked, what you should have reported, how did both these chords get bent?

In my opinion these webs can be brought back to proper line by use of fifteen to twenty 1-inch bolts, threaded at both ends for nuts, passing through the two webs of that half of chord. Of course means must be taken to stiffen the straight web against its bending when the bolts are tightened.

If necessary, after getting the bent webs in line, to hold them, spacers and possibly some through bolts may be used.

Some more satisfactory method than the one shown in your sketch must be devised.

Mr. Deans telegraphs that upon Mr. Szlapka's return he will give me fuller facts.

Yours truly,

THEODORE COOPER.

Then the following telegram was received from Mr. Deans:—

Phœnixville, Pa., August 9, 1907.

Theodore Cooper,
Consulting Engineer,
45 Broadway, New York.

Mr. Szlapka happened to be at bridge site yesterday; expect him home to-morrow, with full information concerning chord joint; will then write you fully.

JNO. STERLING DEANS.

To which Mr. Cooper replied as follows:—

New York, August 9, 1907.

John Sterling Deans,
Chief Engineer Phœnix Bridge Company,
Phœnixville, Pa.

Dear Sir,—Your telegram regarding chord joint at hand. The method proposed as sketched by Mr. McLure is not satisfactory as I telegraphed yesterday.

These bent webs can be pulled back by use of about fifteen to twenty 1-inch bolts (in $1\frac{1}{16}$-in. holes) threaded at both ends for nuts, passing from the outer to the inner bent web. The outer straight web being stayed in some manner against its bending.

If the bent webs after being pulled into line, tend to go back when released from the bolts, stays must be introduced to hold them in position. Possibly it may be necessary to permanently rivet in some of these 1-inch bolts.

Please let me know what method you propose to use.

It is a mystery to me how both these webs happened to be bent at one point and why it was not discovered sooner.

Yours very truly,

THEODORE COOPER.

On August 10, Mr. Deans wrote as follows:—

PHŒNIXVILLE, PA., August 10, 1907.

THEODORE COOPER, Esq.,
Consulting Engineer,
45 Broadway, New York.

DEAR SIR,—Splice cantilever chords 7 and 8.

Mr. Szlapka did not return to-day as expected, but will no doubt be here on Monday, when we will write you at once.

Yours truly,

JOHN STERLING DEANS.

and on 12th Mr. Deans wrote as follows:—

PHŒNIXVILLE, PA., August 12, 1907.

THEODORE COOPER, Esq.,
Consulting Engineer,
45 Broadway, New York.

DEAR SIR,—Chord splice south cantilever arm, 7 L and 8 L.

Mr. Szlapka reached the office this morning and I am able to give you information in conection with this one joint.

All ribs of the chord 7 L have a complete and full bearing on ribs of 8 L. The bend was no doubt put in the rib in the shop, before facing and was probably done when pulling the ribs in line to make them agree with spacing of these ribs and the clearance between ribs, called for on the drawing. The bend being on only one rib of one chord, there being a full bearing over the entire rib, all splice plates being readily put in position, we do not think it necessary to put in the diaphragm suggested by the erection department.

Please let us hear from you on this subject promptly, and oblige.

Yours truly,

JOHN STERLING DEANS.
Chief Engineer.

On August 13th in reply to Mr. Deans, Mr. Cooper wrote as follows:—

NEW YORK, August 13, 1907.

JOHN STERLING DEANS,
Chief Engineer,
Phœnix Bridge Company,
Phœnixville, Pa.

DEAR SIR,—The information regarding chord splice 7 and 8 L, is so different from the dimension sketch sent by Mr. McLure, I can take no action on this matter till the exact facts are presented. Please have your resident engineer and Mr. McLure re-examine this joint and send the exact condition of this rib, as to the amount of the bends and relation of the bearing surfaces to each other.

I don't see how one rib being bent, only, as stated in your letter, there can be a complete and full bearing of these ribs.

Neither can I understand how pulling the ribs into line at the shop could bend it out of line.

I will write Mr. McLure to-day to have a further investigation of this joint and to report as promptly as possible.

Yours very truly,

THEODORE COOPER.

And on the same day Mr. Cooper wrote Mr. McLure:—

NEW YORK, August 13, 1907.

N. R. McLURE, Esq.,
Inspector for Erection, Quebec Bridge,
New Liverpool, P.Q., Canada.

DEAR SIR,—Mr. Deans writes me that only one rib at joint 7 and 8 L is bent, and still that there is a full and complete bearing, that the bend was no doubt put in the chord in the shop before facing.

I have asked him to instruct his resident engineer to join with you in making an exact report, with dimensions of the conditions of this joint, with amount of bearing and if it is a square bearing or askew.

In reference to the splicing of T-5 and T-5 O mentioned in your letter of 10th, I do not care to interfere with the regular programme as I have not followed the various actions of the loadings at different stages. Without going into it carefully, I think there will be more compression at these points with more of the suspended span in place.

Please report promptly regarding joint 7 and 8-L with all the facts.

Yours truly,

THEODORE COOPER.

Mr. Deans wrote Mr. Cooper on 14th as follows:—

PHŒNIXVILLE, PA., August 14, 1907.

THEO. COOPER, Esq.,
Consulting Engineer,
45 Broadway, New York.

DEAR SIR,—Chord splice 7 and 8 L—Your letter August 13th.

I will have a full and complete report made of this joint by Mr. McLure and Mr. Birks and submit it to you earliest possible moment.

Yours truly,

JOHN STERLING DEANS, *C.E.C.*,
Chief Engineer.

On August 14 Mr. Cooper received the following letter of 12th from Mr. McLure:—

NEW LIVERPOOL, P.Q., CANADA,
August 12, 1907.

Mr. THEODORE COOPER,
Consulting Engineer,
45 Broadway, New York.

DEAR SIR,—I beg to acknowledge the receipt of your letter of August 9 and have noted what you say regarding the method of repairing splice between chord 7 and 8 cantilever arm west truss. We will not do anything with this then until the matter has been arranged between yourself and Mr. Szlapka.

The reason I did not report at first as to how these chords got bent was because there were many different theories here as to the cause, no one of which I was at that time ready to accept. One thing I am reasonably sure of, and that is that the bend has occurred since the chord has been under stress, and was not present when the chords were placed. This being the case, the cause of the bend would seem to be the slight overrunning in length of the bent rib in either chord 7 or 8. Owing to the fact that these chords are faced on the rotary machine the four ribs at once, this would at first seem to be out of the question, but it seems to me that after the first end of a chord has been faced in turning it with the crane, to bring the other end into position, for facing, it might be possible for one rib to work slightly by the others longitudinally, without being noticed, and in spite of the latticing and thus cause a slight difference in

length. In fact, in taking the opening in the chord splices on the south anchor arm, it has often been noticed that a considerable variation existed between the openings of the different ribs at the same splice, which difference I was not able to account for except by the above theory that, during transportation, and in the handling before erection, some of the ribs have worked slightly in a longitudinal direction by each other. In the case in question, of course, this must have happened between the time of facing one end and the other. If this is correct, then it will be a pretty hard matter to draw the splice back into line with bolts, and our idea in suggesting that diaphragm was to prevent this eccentricity from increasing, rather than to correct that already there.

As I had supposed, the strike in force for the last three days of last week, has been settled and work was again resumed this morning. A meeting of the 'Union' was held Saturday night and enough of the discontented element had been lost so that when the matter was brought to a vote the majority were found to be in favour of returning to work under the original agreement. Those who were not in favour of returning to work, however, are now leaving so that our force is reduced greatly on both sides of the river.

Since writing the above I have discovered that splice between the chords 8 and 9 on west truss of south cantilever arm is in the same condition exactly as that between 7 and 8, except that the bend is *only* $\frac{5}{16}$-in. instead of $\frac{3}{4}$-in. at the bottom, and runs out so that on top this rib is in line as are the other three.

This is the same rib, and the bend is in the same direction as that reported for the other splice. When it is decided in what way to treat the splice between chords 7 and 8 we will repair that between chords 8 and 9 in a similar manner.

Yours very truly,

N. R. McLURE.

To this Mr. Cooper replied on August 15 as follows:—

NEW YORK, August 15, 1907.

N. R. McLURE, Esq.,
Inspector Erection, Quebec Bridge,
New Liverpool, P.Q., Can.

DEAR SIR,—None of the explanations for the bent chord stand the test of logic.

I have evolved another theory, which is a possible if not the probable one. These chords have been hit by those suspended beams used during the erection, while they were being put in place or taken down. Examine if you cannot find evidence of the blow, and also make inquiries of the men in charge.

Yours very truly,

THEODORE COOPER.

A further report was made by Mr. McLure on August 16:—

NEW LIVERPOOL, P.Q., CANADA, August 16, 1907.

Mr. THEO. COOPER,
Consulting Engineer, 45 Broadway,
New York.

DEAR SIR,—Referring to your letter of the 13th, regarding splice between 8-L and 7-L on south cantilever arm, you have no doubt by this time received my letter of the 21st instant, giving my theory of the cause of this bend. These conditions are as indicated in my report of August 6. Mr. Birks, the resident engineer for the Phœnix Bridge Company, reported exactly the same thing, in somewhat different language to Phœnixville, but Mr. Deans has evidently taken a different meaning from his report

than was intended. He evidently thinks that only one rib of one chord is bent, whereas it is the same rib of each chord, as indicated in the sketch I sent you. There is really nothing to add to the two letters I have already written regarding this bend, except to say that all the four ribs have full bearing on each other, as indicated also in the sketch of August 6. In order to verify our first reports, Mr. Birks and I made a careful and more thorough measurement of this splice to-day, both top and bottom, and I am inclosing a blue print of a sketch made as a result of these measurements. It indicates practically the same condition as described in my first letter, except that it is given more in detail (see drawing No. 30).

As to the cause of this bend, regarding which I wrote you on August 12, Mr. Deans seems to think that it was put in in the shops; but that is because he did not understand the conditions existing. Aside from the fact that it would be hardly probable that these two ribs of different chord sections should be bent the same way, exactly the same amount in the shops to dimensions ½-inch to ¾-inch less than called for; I am reasonably sure, as I said before, that this condition did not exist before the erection of these chords, as I have personally inspected every member yet erected in this bridge thus far, except the bottom chords of anchor arm, on the cars just before the erection, looking particularly for bends in ribs of compression members, and wherever discovered have taken measurements of the amounts and recorded them. If these ribs then had been this much out of line before erecting, it would be well nigh impossible to miss seeing them. Consequently the only way the bend could have occurred, it seems to me, is that reported in my letter of August 12.

I trust that these explanations, with the inclosed sketch, will make the matter entirely clear. Mr. Birks is sending same sketch to Phœnixville to-day.

Yours very truly,

N. R. McLURE.

Mr. Deans also received a copy of this sketch, and wrote Mr. Cooper on August 20 as follows:—

PHŒNIXVILLE, August 20, 1907.

THEO. COOPER, Esq.,
Consulting Engineer, 45 Broadway,
New York.

DEAR SIR,—We have advice from your field that you received copy of sketch No. 28, giving further details in connection with cantilever chord splice 7-L and 8-L. You will notice that the two chords have a perfect bearing with each other at all ribs; both chords having one bent rib and not one chord only as we first understood.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

To which Mr. Cooper replied on August 21 as follows:—

NEW YORK, August 21, 1907.

JOHN STERLING DEANS, Esq.,
Chief Engineer, Phœnix Bridge Company,
Phœnixville, Pa.

DEAR SIR,—I received copy of sketch of joint 7 and 8-L two days ago.

I wrote Mr. McLure last week, telling him none of the theories as to how this bending occurred were logical. That my theory was a blow on this rib after the two sections were in contact, and that it probably was done in moving the suspended beams used in erecting. To examine carefully to see if he could find any evidence of this; he has not yet reported. He did report a similar bend at L-8 and 9 west truss in same rib but of less amount.

I still believe this bend can be partly removed by use of long bolts with threads at each end, outer rib being stiffened to prevent its bending. If it can be pulled nearer straight, stays or bolts must be provided to hold it against future movement.

I cannot consent to let it go without further action as the rivets in the cover splices would not satisfy the requirements to my mind.

Yours very truly,

THEODORE COOPER.

This letter was acknowledged by Mr. Deans on August 23:—

PHŒNIXVILLE, PA., August 23, 1907.

THEO. COOPER, Esq.,
Consulting Engineer, 45 Broadway,
New York.

DEAR SIR,—Joint 7-L and 8-L south cantilever arm. Referring to your letter of August 21, I notice you expect to hear again from Mr. McLure. As soon as you have his report kindly let us hear from you again and oblige.

Yours truly,

JNO. STERLING DEANS.
Chief Engineer.

On August 26 Mr. Cooper wrote the following letter:—

NEW YORK, August 26, 1907.

JOHN STERLING DEANS,
Chief Engineer Phœnix Bridge Company,
Phœnixville, Pa.

DEAR SIR,—Mr. McLure reports that he can find no evidence of the bent ribs having been hit and does not think they could have been struck. This only makes the mystery the deeper, for I do not see how otherwise the ribs could have been bent.

When convenient I would like to discuss with Mr. Szlapka the best means of getting these ribs into safe condition to do their proper work.

Yours very truly,

THEODORE COOPER.

This was acknowledged August 27 by Mr. Deans:—

PHŒNIXVILLE, PA., August 27, 1907.

THEO. COOPER, Esq.,
Consulting Engineer, 45 Broadway,
New York.

DEAR SIR,—Chord splice 7 and 8 cantilever arm, south side.

Replying to your letter of August 26th, I will have Mr. Szlapka call to see you first opportunity, to discuss this question. He will wire you later the day he will be in New York.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

This was the last that transpired with regard to the bent ribs at joint 7-L and 8-L cantilever arm, and it is plainly indicated that no one except Mr. Cooper looked upon this matter as serious or as indicating any consitutional weakness. It will be noted that the bends at 7 and 8 were reported on August 6, the bends at 8 and 9 discovered

on August 12, and that both bends were in the west truss, that previously from time to time chords with ribs more or less wavy had been reported, and Mr. McLure gave it as his opinion that these bends were caused by stress since erection, because he was sure they were straight when erected, while Mr. Deans thought the bends were made in the shop.

While Mr. Deans, after Mr. Szlapka's return, gives certain information as to the bend in the 7 and 8 splice, Mr. Szlapka states that on his visit to the bridge he did not examine this splice, and further says that during none of his three visits to the bridge did he examine any chords.

Mr. Kinloch states in his evidence that he did not notice the bends at the 7-L—8-L joints when the bottom cover plate was first removed, and that he felt confident that these distortions took place after the removal of the cover plate.

It seems clear from the above that Mr. Cooper's statement that the delicacy of the joints was not sufficiently appreciated by the Phœnix Bridge Company is substantiated. Mr. Szlapka was on the ground and made no special examination in the matter, and and Mr. Deans endeavoured to throw the blame for the distortions entirely on the shop work. No evidence has been shown to us to prove that Mr. Deans had any grounds for this assertion, and his inspector, Mr. Morris, was in possession of information that indicated that there was no great probability that such an error could have escaped detection. On August 20 Mr. Kinloch discovered that chord 8-R of cantilever arm was bent, and afterwards found that 9-R and 10-R also showed distortion, he called Mr. Birk's attention to this condition, but neither of them considered it of importance. Mr. McLure was ill and did not see these bends until several days after they were found (August 23), but Mr. Yenser was made aware of them. On August 23 the joint at chords 5-6 R of cantilever arm was found to be off on one centre rib ½-inch at bottom, the offset running to nothing at top. Mr. Kinloch visited chord 8-R daily for several days and imagined that the bend was becoming greater, all four ribs being bent, but not alike.

The bend in chord 9-L anchor arm was discovered about 9.30 a.m., August 27, to have greatly increased, it having been previously noted and being under observation. Owing to the fact that the 25th was a Sunday, and that there was practically no work done on the 26th, it is doubtful whether this chord was examined between the 24th and the 27th. Mr. Kinloch, who made the discovery, in his evidence says :—

'Q. Please relate the occurrences following your discovery of the bent chord on August 27?

'A. Immediately after discovering the bend I brought the matter to the attention of Mr. Yenser and Mr. Birks, and with them re-examined both chord A 9-L and several other lower chord members. We did not know what to make of the matter, and then went up to our office and arranged with Mr. McLure to have the deflections of the suspicious chords measured. This measurement, which was made by Birks, McLure and myself, showed the extent of the deflections; and their cause and their ultimate result immediately became a matter of very active discussion. Mr. Birks expressed himself definitely as being of opinion that there was no danger, and endeavoured to persuade me that the bend had always been in the chord. Mr. Yenser and I were uneasy, and considered the matter serious, and finally suggested that Mr. McLure and Birks should go to New York and Phœnixville for advice. It was considered that the matter could not be satisfactorily explained by telegraph or telephone, and none of us expected immediate disaster. Mr. Birks and Mr. McLure did not welcome our suggestion, saying that they would only be laughed at on arrival, and it was finally agreed to refer the matter of sending to headquarters to Mr. Hoare, who decided in favour of our suggestion. Mr. Hoare visited the bridge on the Wednesday and spent most of the day there. He appeared very anxious that I should abandon my position of being positively convinced that the bend had occurred since the erection of the cantilever arm was completed, and argued both this and some

possible methods of strengthening the chords by bracing several times with me. I was somewhat excited and much annoyed at the unwillingness of all the engineers to accept my statement of facts, and on both Wednesday and Thursday avoided further discussion of the matter as much as possible. It was understood that Mr. McLure would immediately wire me if Mr. Cooper took a serious view of the situation, but this he failed to do. Mr. Birks, however, told me on the morning of the 29th instant that he had been advised by 'phone from Phœnixville that they had a record which showed that the bends had been in the chord before it was shipped from Phœnixville, and that he had just advised Mr. Hoare by telephone at the request of Mr. Deans to that effect.'

As soon as the measurements above referred to were made, it was recognized by Mr. Yenser and the inspectors that they were face to face with a crisis. Mr. Yenser announced his intention of stopping erection until he had referred the matter to Phœnixville. The measurements were plotted (drawings Nos. 28, 29 and 30 have been prepared from these plottings), and were reported by mail to Mr. Cooper and to Phœnixville, these reports being delivered on the morning of the 29th. Owing apparently to anxiety already existing among the workmen (see evidence D. B. Haley) it was not considered wise to use either telegraph or telephone. As suggested by Mr. Kinloch, Mr. McLure reported the matter fully to Mr. Hoare on the evening of the 27th, the delay of about twelve hours being accounted for by the making and plotting of the measurements and the necessity of using a personal messenger, as it was not wished to report particulars over the telephone. It is clear that Mr. Yenser, Mr. Kinloch and Mr. McLure were very much alarmed, but Mr. Birks could not be convinced that the bends had recently taken place. He knew better than anyone else on the work the care with which the calculations and designs had been made, he was familiar with the experience and abilities of the designers, and could calculate that the stresses were then far below the expected maximum. To engineers the force of such reasoning is very great, and we do not consider that the confidence Mr. Birks placed in his superiors was in any way unusual or unreasonable. There was no misunderstanding, however, on his part; he realized that if the bends had not been in the chord before it was erected the bridge was doomed, and although Mr. McLure had evidence that the bends had increased more than one inch in the course of a week, although Mr. Kinloch was positive that the bends had very recently greatly increased, and although Mr. Clark stubbornly maintained that the chord was absolutely straight when it left Chaudière yard, Mr. Birks still strove to convince himself that they must have been mistaken. Mr. Hoare evidently concluded that the matter was too serious for him to settle by any offhand decision, and approved Mr. McLure's mission to New York, wisely requiring that he should get all possible facts before leaving, so that Mr. Cooper need not wait for further information on which to base a decision.

The text of Mr. McLure's report of August 27th is as follows:—

New Liverpool, P.Q., August 27, 1907.

Mr. Theodore Cooper,
Consulting Engineer,
45 Broadway, New York.

Dear Sir,—I inclose sketches showing condition of bottom chord sections No. '606-9 L' of south anchor arm and '621-9 R and 8 R' of south cantilever arm, as found from measurements made to-day by the Phœnix Bridge Company's assistant engineer and myself, by stretching a line from batten plate to batten plate as indicated on the sketches and measuring from this line held taut, to each rib, top and bottom. It was noticed this morning that these chords were bent in this manner, as it is very evident to one walking over them, and as it looked like a serious matter, we measured them.

Although a number of the chords originally had ribs more or less wavy, as I have reported to you from time to time, it is only very recently that these have been in this condition, and their present shape is undoubtedly due to the stress they are now receiving. Only a little over a week ago, I measured one rib of the 9-L chord of anchor arm here shown, and it was only ¾-inch out of line. Now it is 2¼ inches.

In the sketches the red indicates straight lines, and black ones the ribs of chords. A top and bottom view is shown in each case. You will note that chords '606-9 L' and '621-9 R' have all ribs bent in same direction, while '621 8-R' has its ribs bent in reverse curves. These bends had become so apparent by to-day that the gangs riveting at these points noticed them, and called Mr. Kinloch's attention to them.

This matter is being reported in this mail, with sketches from the same measurements, to the Phœnixville office, and the erection will not proceed until we hear from you and from Phœnixville.

Yours very truly,

N. R. McLURE.

Wednesday, August 28, was a day of waiting and uncertainty. Mr. Yenser had changed his mind during the night and in the morning continued erection. The men were uneasy and alarmed and the officials were anxiously awaiting instructions from Phœnixville or New York. Mr. Yenser's decision to continue work, was laid before Mr. Hoare, and Mr. Hoare, upon whom, as chief engineer, the final responsibility for every step taken rested, decided that he had acted wisely. Mr. Hoare makes this clear in the following letters to Mr. Cooper:—

Letterhead—

(The Quebec Bridge and Railway Company.)

QUEBEC, August 28, 1907.

THEODORE COOPER, Esq.,
35 Broadway,
New York City.

DEAR SIR,—I wired you to-day as under:—

Have sent Mr. McLure to see you early to-morrow to explain letter mailed yesterday about anchor arm chords.

Also the following message to the Phœnix Bridge Company. 'Mr. McLure will call to-morrow to explain Birks' letter *re* anchor arm chords, will see Mr. Cooper first.'

Regarding this matter I thought it best for McLure to go at once to be able to explain matters and answer questions. He did not have much time for extended investigation before leaving.

I have been at the bridge all day trying to get some evidence in connection with the bending of the ribs in this chord. Mr. Kinloch noticed it for the first time yesterday and all inspectors declare that no such pronounced distortion existed a few weeks ago. Mr. McLure made measurements yesterday afternoon and brought them to my house late last night, and stated that the erection foreman hastily concluded that he would not continue erecting to-day, which alarmed me at the time. Upon arriving at the work this morning he thought better of it and decided to go ahead, at the same time asking me if it would be all right. After ascertaining that the effects from moving the traveller ahead and proceeding with the next panel would be so insignificant I requested him to continue, as the moral effect of holding up the work would be very bad on all concerned and might also stop the work for this season on account of losing the men. From further investigation during the day I cannot help concluding that the metal received some injury before it was erected, as the corresponding chord in the same panel, and stressed the same, is in good condition. These panels are being stressed to-day, approximately, about $\frac{7}{10}$ths of their maximum, and it is difficult to believe that this is the entire cause of the distortion. Now and again a rib in certain

members is found to be a trifle longer than another, which, when compressed, might cause a trifling kink in it. There are a few examples of this. The chord in question, when being lifted to the cars in the storage yard broke loose from the grips, one end of which fell a distance of 6 feet on to timber sills, the other end fell a distance of 2 feet on to a block of eyebars. In falling it fell over on its side breaking one of its angles on the north end splice and twisting some of the lacing bars, all of which were renewed. After this the inspectors reported the ribs perfectly straight. On account of this chord falling on to two rigid higher points at ends, with no support in the middle but soft material, the conclusion would be that the deflection would be downward; as a matter of fact, the evidence shows that it was in the opposite direction. Since Mr. McLure left, Mr. Birks has made careful examination of the chord and states that the actual bending commences at the south splice and was not confined entirely to the lengths between the batten plates, where the lacing angles are used. As the foreman and inspectors declare that these defects were not noticeable until recently, perhaps the stress in this chord has made previous defects more pronounced. I thought I would give you the above story from further investigation by to-night's mail to help you come to some conclusion.

Yours truly,

E. A. HOARE.

(Letterhead, the Quebec Bridge and Railway Company.)

QUEBEC, August 29, 1907.

THEODORE COOPER, Esq.,
35 Broadway, New York City.

DEAR SIR,—Mr. Birks has just called me up on the telephone from the bridge, and states that he has received a message from Phœnixville stating that they have positive evidence that the chord was not straight before it left the shops. This possibly clears up the mystery why the deflection was in the opposite direction to what it should have been, due to its fall in the storage yard. Mr. Birks has wired that information to Mr. McLure at your office. Mr. Birks further stated that he is positive that the chord ribs were more or less out of line when the splice at the south end was riveted up in the bridge.

Yours truly,

E. A. HOARE.

(Letterhead, the Quebec Bridge and Railway Company.)

QUEBEC, September 2, 1907.

THEODORE COOPER, Esq.,
45 Broadway, New York City.

DEAR SIR,—I thank you for replies to all our messages. I am sorry that you are not well, and of course this appalling disaster has made you feel a thousand times worse.

Mr. Berger will answer our purpose very well for the present. The investigating commission may find it necessary later to interview you in New York, due notice of which will be given you.

I wish to correct a misstatement in my letter to you of the 28th August, which was written late and very hastily, to confirm telegram and conversation with Mr. Birks about the chord under discussion. The statement in my letter, as follows:—

'Mr. McLure made measurements yesterday afternoon, and brought them to my house late last night, and stated that the erection foreman hastily concluded that he would not continue erecting to-day, which alarmed me at the time. Upon arriving at the work this morning he thought better of it, and decided to go ahead, at the same time asking me if it would be all right. After ascertaining that the effects from moving the traveller ahead and proceeding with the next panel would be so

insignificant, I requested him to continue, as the moral effect of holding up the work would be very bad on all concerned, and might also stop the work for this season on account of losing the men,'
is to some extent a misstatement of facts and not clearly stated, due to too much haste, and which I wish now to correct as under:—

'Upon arriving at the work that morning the foreman told me that he had considered it during the night, and had already moved the traveller forward, asking myself, Mr. McLure and Mr. Birks if we thought that what he had done would do any harm. We all thought that it would not, as they stated it would only add 50 pounds to the square inch to the chord in question. We all thought at the time that to discontinue the work would entirely stop the work for this season, as the men would not wait and would go elsewhere to prepare for the winter. As stated in my last letter, strictly speaking, I did not request the foreman to continue the work, as he had already done so; at the same time we thought there was no immediate danger in adding so small a load. This latter more clearly states the conversation between us, and I am sorry that I have misstated, in my hurry, one or two points which would be more or less confusing.

Yours truly,

E. A. HOARE.

It was clear that on that day the greatest bridge in the world was being built without there being a single man within reach who by experience, knowledge and ability was competent to deal with the crisis. Mr. Yenser was an able superintendent, but he was in no way qualified to deal with the question that had arisen. Mr. Birks, well-trained and clear headed, lacked the experience that teaches a man to properly value facts and conditions; and Mr. Hoare, conscious that he was not qualified to give judgment, simply assented to the courses of action that had been determined on by Messrs. Yenser and Kinloch and made no endeavour to make a personal examination of the suspected chords.

Some measurements were made to test the stability of the main pier, but no one seems to have thought of testing the span for alignment or levels, and, above all, to measure the chords again to see if they showed any increase of deflection. Mr. Hoare discussed some means of bracing the chords, but decided to postpone action until Mr. Cooper was heard from. At Mr. Hoare's request, Mr. Birks inspected the chord A 9-L and the A-L 8-9 joint carefully and his observations tended to reassure both Mr. Hoare and himself, as he thought, that he found evidence of original crookedness in the chord.

His report to Phœnixville which was received on August 30 reads as follows (Exhibit 58):—

New Liverpool, August 28, 1907.

The Phœnix Bridge Company,
Phœnixville, Pa.

Dear Sirs,—I have made a further investigation of chord 9 A, and beg to report following additional data. The bend in the chord starts at the faced splice at the shore end and not at the edge of the splice batten. It appears from this that at least a large portion of the bend was in the chord when the top and bottom splice battens were riveted early in June. This and the fact that the lacing angles are not disturbed leads me to believe that the ribs were bent before erection in spite of the fact that Mr. Clark and Kinloch think all ribs were straight when the chord was repaired. From the evidence so far, I do not think we are justified in assuming it to be a fact that the ribs of any of the chords have buckled since erection, and Mr. Yenser has come to the same conclusion.

Yours truly,

A. H. BIRKS.

After he had made his examination, Mr. Birks called Mr. Kinloch and waited at track level, while Mr. Kinloch went down to the chord and checked Mr. Berks' observations. After careful discussion with Mr. Kinloch of what was then done we are forced to conclude that the sketch in Mr. Birks' letter shows only his personal idea of the shape and extent of the existing distortion and cannot be considered as furnishing data on which to base engineering conclusions, as no actual measurements were taken.

On August 29 Mr. Birks' report of the 27th inst. was received at Phœnixville and was immediately discussed by Messrs. Deans, Szlapka and Milliken. It was finally decided that it was safe for the work to proceed and a telephone conversation took place between Messrs. Milliken and Yenser and another between Messrs. Deans and Birks. Mr. Szlapka had made some calculations and Mr. Birks reported his observations of August 28. Messrs. Yenser and Birks were assured that the office approved their action in continuing work of erection and Mr. Birks was told to tell Mr. Hoare that the bends had been in the chords before they left Phœnixville. This Mr. Birks did.

Mr. Deans also telegraphed Mr. Hoare as follows:—

PHŒNIXVILLE, PA., August 29, 1907.

E. A. HOARE, Esq.,
Chief Engineer Quebec Bridge Company,
Quebec, Canada.

'McLure has not reported here ; the chords are in exact condition they left Phœnixville in and now have much less than maximum load.'

Mr. Hoare had telegraphed to both Mr. Cooper and Deans on August 28, advising them of Mr. McLure's mission. Mr. Deans has since explained that his telegram did not refer to the chords measured on the 27th inst., but after considering the circumstances we are entirely satisfied that Mr. Hoare was justified in thinking that it did, and in so doing he was confirmed by Mr. Birks' telephone message previously received.

From the time that these assurances were received, anxiety at the bridge practically ceased, and there is no evidence that any further measurements were made to determined the movements of the suspected chords. As Mr. Hoare expressed it, 'I felt quite comfortable that day about it. I knew it could not be long before the matter would be taken up.'

Shortly after 11 a.m. on August 29 Mr. Cooper reached his office and found Mr. McLure there. After a brief discussion Mr. Cooper wired to Phœnixville as follows:—

NEW YORK, August 27, 1907.
12.16 p.m.

PHŒNIX BRIDGE COMPANY,
Phœnixville, Pa.

Add no more load to bridge till after due consideration of facts. McLure will be over at five o'clock.

This message was received at Phœnixville at 1.15 p.m. Mr. Cooper has explained in his evidence that he was not aware at the time that erection was proceeding, Mr. McLure having advised him to the contrary, and that he telegraphed to Phœnixville instead of to Quebec because he thought action would be more promptly secured by so doing.

Mr. McLure had promised to wire Mr. Cooper's decision to Mr. Kinloch immediately, but he did not do so.

Mr. Deans reached his office about 3 p.m., and found Mr. Cooper's telegram there. He arranged for Mr. Szlapka and Mr. Milliken to be on hand to meet Mr. McLure, but otherwise took no action. After Mr. McLure arrived there was a brief discussion, during which Mr. McLure mentioned that he had received a wire from Mr. Birks giving him the result of that gentleman's observations on August 28. It was decided to postpone action until the morning, and to await the arrival of Mr. Birks' letter of August 28. This decision was made almost at the minute that the bridge fell.

As a conclusion reached from the evidence and from our own studies and tests, we are satisfied that the bridge fell because the latticing of the lower chords near the main pier was too weak to carry the stresses to which it was subjected; but we also believe that the amount of those lattice stresses is determined by the deviation of the lines of centre of pressure, from the axis of the chords, and this deviation is largely affected by the conditions at the ends of the chords. We must, therefore, conclude that although the lower chords 9-L and 9-R anchor arm, which, in our judgment, were the first to fall, failed from weakness of latticing; the stresses that caused the failure were to some extent due to the weak end details of the chords, and to the looseness, or absence of the splice plates, arising partly from the necessities of the method of erection adopted, and partly from a failure to appreciate the delicacy of the joints, and the care with which they should be handled and watched during erection. We conclude from our tests that owing to the weakness of the latticing, the chords were dangerously weak in the body for the duty they would be called upon to do. We have no evidence to show that they would have actually failed under working conditions had they been axially loaded and not subject to transverse stresses arising from weak end details and loose connections. We recognize that axial loading is an ideal condition that cannot be practically attained, but we do not consider that sufficient effort was in this case made to secure a reasonable approach to this condition. The Phœnix Bridge Company showed indifferent engineering ability in the design of the joints, and did not recognize the great care with which these should be treated in the field.

We consider that Mr. Deans was lacking in judgment and in sense of responsibility when he approved of the action of Mr. Yenser in continuing erection, and when he told Mr. Birks and Mr. Hoare that the condition of the chords had not changed since they left Phœnixville.

No evidence has been produced before the commission in proof of the correctness of this statement about the chords, and Mr. Szlapka's calculations as stated in the following letter showed that the rivets were even then loaded to their maximum specified stress of 18,000 pounds per square inch.

MONTREAL, January 24, 1908.

Messrs. PHŒNIX BRIDGE COMPANY,
Phœnixville, Pa.

GENTLEMEN,—Will you please file with the commission a copy of the calculations made by Mr. Szlapka on August 29, 1907, and which are referred to on pages 967 and 968 of the evidence.

As we are nearing the completion of our report, we would esteem it a favour if you would have this information sent to us immediately.

It is possible that you may not have an exact copy of these calculations, but no doubt they can be duplicated, and Mr. Szlapka's certificate to this effect will be sufficient.

Yours truly,

HENRY HOLGATE.

PHŒNIXVILLE, PA., January 31, 1908.

HENRY HOLGATE, Esq.,
Chairman, Royal Commission,
Montreal, Canada.

DEAR SIR,—Replying to your letter of January 24, I inclose herewith letter from Mr. Szlapka of this date, giving calculations similar to that made on August 29, regarding chord 9-L south cantilever arm.

Yours truly,

JNO. STERLING DEANS,
Chief Engineer.

PHŒNIXVILLE, PA., January 31, 1908.

JOHN STERLING DEANS, Esq.,
Chief Engineer, The Phœnix Bridge Company,
Phœnixville, Pa.

DEAR SIR,—Referring to Mr. Holgate's letter of January 24 addressed to the Phœnix Bridge Company, I beg to give you below the calculations similar to the one made on August 29, 1907, referring to chord 9-L south anchor arm.

Taking $1\frac{1}{2}$-inch as the average reported curvature of chord 9-L we have:—

$$\frac{W\,L}{4} \times 12 = 780'' \times 18{,}000 \times 1\tfrac{1}{2}\text{-inch} = 21{,}060{,}000 \text{ inch lbs.}$$

$$\frac{W}{2} = 61{,}600 \text{ lbs.}$$

$$\text{Stress in each lattice } S = \frac{61{,}600 \times 1{\cdot}4}{4} = 21{,}600 \text{ lbs.}$$

Yours truly,

THE PHŒNIX BRIDGE COMPANY.
Per P. L. SZLAPKA.

The theory underlying these calculations is very questionable, but it was adopted in the design of the bridge (See Appendices Nos. 16 and 17) and we cannot understand why its warning was so entirely disregarded in the face of the consequences that might result.

With reference to Mr. Cooper's telegram, Mr. Deans knew that he was in possession of later information from the bridge than had reached Mr. Cooper and therefore decided to wait for Mr. McLure and afterwards for the arrival of Mr. Birks' letter of August 28 before taking action. The whole incident points out the need of a competent engineer in responsible charge at the site.

Mr. Hoare was the only senior engineer who was able to reach the structure between August 27 and August 29. He was fully advised of the facts yet did not order Mr. Yenser to discontinue erection which he had power to do; we consider that he was in a much better position than any other responsible official to fully realize the events that had occurred, and his failure to take action must be attributed to indecision and to a habit of relying upon Mr. Cooper for instructions.

We are satisfied that no one connected with the work was expecting immediate disaster, and we believe that in the case of Mr. Cooper his opinion was justified. He understood that erection was not proceeding; and without additional load the bridge might have held out for days.

Our tests have satisfied us that no temporary bracing such as that proposed by Mr. Cooper could have long arrested the disaster; struts might have kept the chords from bending, but failure from buckling and rivet shear would soon have occurred.

The following drawings may be consulted in connection with this Appendix:—

Drawing No. 1. General plan of site and vicinity.
" 2. General dimensions of bridge members.
" 5. Erection marks on bridge members.
" 7. Dates of riveting.
" 9. Sections of bridge members and erection stresses.
" 10. Plan showing positions of eye witnesses.
" 13. Loading of bridge on August 29.
" 28, 29, 30. Chord bends measured on August 6, 12, and 27.
" 36. Detail of chord No. A-9.

Our conclusions are based to a considerable extent on the facts set forth in the following appendices:—

Appendix No. 13. Full sized column tests.
" 15. Special tests made in connection with the Quebec Bridge.
" 16. The theory of built up compression members.
" 17. A comparison of chord designs.
" 18. A discussion of the specifications.

LIST OF ORDERS FOR QUEBEC BRIDGE.

Series D. *Anchorage.*

C.O. 700. Eyebars and pins.
" 701. Plate girders and I-beams.

Approach Spans at each end of Bridge.

C.O. 702. 2-210′ 0″ C. of E. pins deck spans for D. Tr. Ry. 2 roadways and 2 sidewalks.
" 703. 2 bents for above spans, about 50′ high.
" 704. 3 full sizes test eyebars.
" 705. Anchorage for 1-214′ south approach span.

Series E. MAIN RIVER BRIDGE.

C.O. 600. Sundry field charges, such as rents, watching, engineering work, &c.
" 601. Field plant charges: steel traveller, tools, engines, rope, blocks, cars, boats, &c., and only such field labor as is used in making tools.

Anchorage.

C.O. 602. Eyebars and pins for anchorage for south approach.
" 603. " " north approach.
" 604. Towers and bracing for south anchorage.
" 605. " " north anchorage.

2,500-foot Anchor Arms.

" 606. Trusses and bracing for south anchor arm.
" 607. " " north anchor arm.
" 608. Eyebars for trusses for south anchor arm.
" 609. " " north anchor arm.
" 610. Pins for trusses for south anchor arm.
" 611. " " north anchor arm.
" 612. Centre posts and bracing for south pier.
" 613. " " north pier.
" 614. Shoes and pedestals for south pier.
" 615. " " north pier.
" 616. Plate floorbeams, stringers and bracing, south anchor arm.
" 617. " " " north anchor arm.
" 618. Trussed floorbeams for south anchor arm.
" 619. " " north anchor arm.
" 620. Full size test eyebars for C.O. 602 and 603.

2,562-foot 6-in. Cantilever Arms.

" 621. Trusses and bracing for south cantilever arm.
" 622. " " north cantilever arm.
" 623. Eyebars for trusses for south cantilever arm.
" 624. " " north cantilever arm.
" 625. Pins for trusses for south cantilever arm.
" 626. " " north cantilever arm.
" 627. Plate floorbeams and stringers and bracing, south cantilever arm.
" 628. " " " north cantilever arm.
" 629. Trussed floorbeams for south cantilever arm.
" 630. " " north cantilever arm.

675-foot Suspended Span.

" 631. Trusses and bracing for southern half of suspended span.
" 632. " " northern half of suspended span.
" 633. Eyebars for southern half of suspended span.
" 634. " northern half of suspended span.
" 635. Pins for southern half of suspended span.
" 636. " northern half of suspended span.
" 637. Floorbeams and stringers for south half of suspended span.
" 638. Floorbeams and bracing for north half of suspended span.

HENRY HOLGATE,
Chairman.

J. G. G. KERRY,
J. GALBRAITH.

APPENDIX No. 12.

A DESCRIPTION OF THE FALLEN STRUCTURE.

The Commission began its inquiry by examining a number of workmen who were understood to have seen the disaster.

A study of this portion of the evidence brings out clearly one or two main facts but with an almost complete absence of important detail. This is not surprising, when the suddenness with which the disaster came and the few seconds occupied by the downfall are considered. The evidence of Huot, who ran from the second panel of the anchor arm to the office at his topmost speed, enables us to fix the duration of the fall at not above 15 seconds. The distance is almost 100 yards, and the floor was already opening between the end of the anchor arm and the approach span as he passed that point. It is not surprising that accurate evidence was not obtainable, as every man's first thought was of self-preservation, and there was no time to consider or realize what was happening.

The records of the inspectors, which show the deformations that were taking place during the month preceding the accident, are corroborated by the witnesses, D. B. Haley and Alexandre Beauvais, the latter in particular testifying to the 'working' of the ribs both at joint A 9-10 R and at joint A 9-10 L. It should be noted that neither of these joints gave way at the time of the accident, and that injuries that they have received are due to the fall itself.

The collapse came very suddenly. The witnesses who were on the bridge outside of the main pier, Haley, Nance, Hall, Davis and Laberge, all testify that they had no warning of any kind, and several of the men who were working on the ground directly under the anchor arm, were caught by the falling structure and killed, when by moving not more than 50 feet they would have saved themselves.

The cantilever arm and suspended span fell as a whole. The witnesses Wickizer and Esmond, both of whom were in good position for observation (see drawing No. 10), testify to the whole cantilever arm falling as one piece, and the former adds that the outer end of the cantilever arm swung slightly to the east, so that he could see directly between the trusses from his position on the jetty on the north shore.

The big traveller fell as if it was part of the cantilever arm, and did not upset, at least until after the arm had struck the waters. The accident was immediately

followed by the rising of a cloud of dust and spray that obscured everything, and there is no evidence concerning the final movement of the traveller; the witnesses Hall and Laberge, testify that it did not upset, to their knowledge. The tops of the centre posts moved slowly riverwards and dropped suddenly, when the centre post feet kicked off the pedestals on the main pier; the post feet were forced southwards. The witness Chase states that he saw these movements.

The anchor arm broke near the centre, and in its first movement appeared to rise in the neighbourhood of the break, and then fall; the witness Culbert states that he observed this.

James Johnson testified that he thought the lower chord of the anchor arm near the third panel from the main pier struck the ground first; and Delphis Lajeunesse, clinging to the west truss of the anchor arm as it fell, noticed that the trusses appeared to be tippling over towards the east.

The anchor arm fell almost without movement to the right or to the left. Mr. Kinloch noted that the portal posts sank down and, using his own simile 'as if they were ice pillars whose ends were rapidly melting away.' In other words, as he stood near the centre line of the track and opposite the office, the end posts while falling straight away from him, appeared to only settle down.

Mr. Cudworth's evidence indicates that the trusses first tippled slightly to the east, he being able to see only the top portions of the centre posts and the adjoining members, then followed an outward movement similar to that described by the witness Chase, and finally everything disappeared suddenly from sight.

Out of eighty-six men on the work only eleven escaped with their lives.

The Commission commenced its examination of the wreck by instructing the inspectors and engineers of the Phœnix Bridge Company and of the Quebec Bridge Company to go over the debris of the anchor arm and to paint in large letters on each main member its erection mark (See drawing No. 5).

The wrecked structure in places was in so chaotic a condition that even these men, who had been familiar with the appearance of every piece of the anchor arm for nearly two years, had difficulty in identifying many of the members.

The photographs, twenty-four in all, that are filed as Exhibit 34, were taken as soon as the marking was completed.

Surveys of the wreckage and adjoining ground were arranged for, the results of these surveys being shown on the following drawings :—

Drawing No. 10.—Plan showing position of witnesses at the time of the fall.

Drawing No. 14.—Check measurements to determine whether any movement of the main pier had taken place.

Drawing No. 15.—Positions occupied by camera when the photographs in Exhibit 34 were taken.

Drawing No. 16.—Diagram of fall—east truss.

Drawing No. 17.—Diagram of fall—west truss.

Drawing No. 18.—Diagram of fall—floor beams and stringers.

Drawing No. 19.—Diagram showing the shape of chords A 9-L and A 9-R after the accident.

Mr. Walter J. Francis, M. Can. Soc. C.E., was requested to make an examination of the wreckage and to prepare such descriptions and photographs of selected bridge details as would be of service in assisting the work of the Commission.

Twenty-three photographs taken by Mr. Francis are filed as Exhibit No. 124.

A number of photographs from Mr. Kinloch's collection are filed as Exhibit No. 35. These photos. show clearly the details of several intricate intersection points, and give an excellent idea of the demands that this bridge made upon the technical skill of the designing officers and upon the resources of the manufacturers. When examining these photographs it should be remembered that the component parts of the structure were never put together until finally erected; every detail was planned

by the designers and made without trial or fitting, prior to erection in place. Several photographs of portions of the wreck are included in Exhibit 35, these having been made by Mr. Kinloch at the request of the Commission.

The measurements to the piers showed that the masonry of the main pier had risen very slightly when relieved of the load of the superstructure; otherwise it had remained exactly in its original position. The results of these surveys were accepted as proof that there were no defects in the substructure or foundations to contribute to the disaster. (Drawing No. 14.)

The plans of the wreckage (Drawings Nos. 16, 17 and 18) show:

(1) That there was practically no lateral movement of the anchor arm, lower chords and floor system while falling. This we regard as a proof of simultaneous failure in the two trusses.

(2) That there were opposite longitudinal movements of those lower chords and parts of floor system that were to the north and south respectively of the joint 8-9 anchor arm. (See Drawings 16 and 17.) This is proof that the initial failure took place close to this joint.

(3) That there was an almost complete destruction of the chords 9 A-L and 9 A-R, that of 9 A-R being the more striking and peculiar. Views of these chords are given in photos Nos. 3, 11 and 12, in Exhibit 34, and in Nos. 18, 19, 20, 21 in Exhibit 35, but their condition after the accident will be more fully realized by reference to Drawings Nos. 18 and 19.

We cannot describe the failure better than by quoting the evidence of Mr. Kinloch, whose knowledge of the structure both before and after its fall is exceptional. (See Evidence.)

'Q. Please describe the movements that you think took place when the bridge was falling?—A. The initial failure I think occurred in both lower chords No. 9 anchor arm simultaneously and in the latticed portion of the chords but not in the same way in both chords. No. 9-L which had previously been observed to be bent deflected slowly and transferred some of its load to 9-R until that chord burst with a sudden fracture accompanied by the loud report testified to by some witnesses. The sudden and complete collapse of 9-R, whilst 9-L was slowly yielding, accounts for the slight swing of the cantilever arm downstream, and for the tendency of the upper portions of the anchor arm to fall in the same direction. At the moment of collapse the thrust of the cantilever arm forced the feet of the main posts off the pedestals and the shoes of the main posts were the first part of the structure to strike the ground. Whilst they were in the air the extremities of the stub chord on the cantilever arm struck the inside coping of the main pier a glancing blow. When the shoes struck the ground that part of the C.P. 6 above the batten plates failed and simultaneously the horizontal strut connecting the two shoes was destroyed. The transverse diagonal bracing between the two posts at the bottom remained intact for an instant and almost the entire weight of the main post and of the top chord was concentrated upon it, causing the bracing to act as a toggle and to force the shoes and the feet of the main post out sideways. This is shown by the holes made in the ground. This action threw the bottom portions of the centre post out of the vertical and permitted the feet of the P-4 posts with the broken ends of A-8 attached to them to pass inside the centre posts, some part of P-4-L striking C.P. 6-L heavily as it fell. During the fall chords 10-R and L cantilever arm which had probably broken loose when the stub chords struck the pier rested for a moment on top of the pedestals and were then partially upended and thrown over on their sides, as they now lie on top of the pier, by the wreckage of S.P. 5 and of the pieces connected to it. Chords 9 of the cantilever arm did not strike the pier before they reached the ground although they now lie with their ends just against the face of the masonry which is slightly marked. Chord 9-R of the cantilever arm is lying in the water with its two inner ribs practically straight and its two outer ribs buckled back in a V-shaped loop about 18 or 20 inches long at a point about 20 feet

from the shop splice, the ends being parallel to the inner ribs. Chord 9-L is buckled at about 15 feet from the field splice in all four ribs to a shape similar to that shown by A-1-R but with a smaller deflection ?

The warnings of coming disaster are thus referred to by Mr. David Reeves. (See Evidence): When the compression members began to yield at several places one after another as we can now see, and the whole bridge was at the verge of collapse, as afterwards developed, &c., &c.' This statement calls for comment.

We do not consider that any of the difficulties with lower chord members noted previous to August 1, 1907, were of serious moment with the possible exception of the fall of chord A-9-L in the storage yard ; the effect of that fall upon the latticing of the member was not determined, and in fact was practically impossible of determination.

Our investigations at Belair yard have convinced us that the several discrepancies noted in the chords during the earlier stages of erection were probably due to errors of shop work and, as Mr. Cooper said, were not serious. The waviness of the ribs which was often recorded by the inspectors might not produce serious results, its importance being dependent upon the strength of the latticing. (See Appendix No. 11). The presence of these bends would materially increase the stresses in the latticing, but we have no evidence to show that there was exceptional waviness in the chords that afterwards deflected most seriously.

The erection of the suspended span did not begin until the middle of July, 1907, and the building out of this span was accompanied by a rapid increase of the stresses in the anchor and cantilever arms. On the day of the disaster the most heavily stressed members (see drawing No. 13) were as follows :—

Member.	Panel No.	Arm.	Stress.
Upper Chord	7	Anchor	17,200 lbs. per sq. in.
"	8	"	17,230 "
"	9	"	18,200 "
"	10	"	18,200 "
"	7	Cantilever..	18,850 "
"	8	"	18,920 "
"	9	"	18,110 "
"	10	"	18,100 "
Lower Chord	5	Anchor	17,010 "
"	6	"	17,100 "
"	7	"	18,040 "
"	8	"	17,830 "
"	9	"	17,910 "
"	10	"	17,560 "
"	7	Cantilever.	17,730 "
"	8	"	17,430 "
"	9	"	17,880 "
"	10	"	17,080 "
Main Diagonal.............	5	Anchor	17,080 "
"	4	"	17,160 "

By the beginning of August the effect of these growing stresses on the weak end details of the chords became perceptible, and by the middle of August the chords began to show signs of failure in the body. On August 6 the deflection of joint 7-8-L cantilever arm was noted, and Mr. Kinloch has expressed his conviction that this deflection occurred after the lower cover plate was removed. The design of the chord ends and joints was such that it is probable that Mr. Kinloch's conclusion is correct, and that

the removal of the lower cover plate weakened the joint appreciably. Any distortion at the joint would throw considerable stresses into the latticing unless the batten plates were of great strength and stiffness. On August 12 the inspectors reported a similar deflection at joint 8-9-L cantilever arm. On August 8 the workman, Haley, noticed that the ribs at joint 8-9-R, cantilever arm, did not match properly, and on August 28 he noticed that the splice plates were bulging. This was noticed by Mr. Kinloch also, who was confident that they were all right when rivetted. Haley also saw that chord 8-R, cantilever arm, close to the joint just mentioned, was bending in all its ribs. The workman, Beauvais, noticed, during the two weeks previous to the accident, that the inner ribs at the joint 9-10-R, anchor arm, were gradually coming together, but does not seem to have reported this. About August 20 Mr. Kinloch noticed that chords 8, 9 and 10-R, cantilever arm, were wavy in the body, but was not sure whether the bends were shop bends or not ; he consulted Mr. Birks and they agreed that these waves were of no importance. On August 25 the deflection at joint 5-6-R, cantilever arm, was discovered. On August 27 the bending in chords 9-L, anchor arm, and chords 8 and 9-R, cantilever arm, had become so noticeable that they were measured in detail, and reported to both headquarters. Mr. McLure's records note a deflection of $\frac{3}{4}$-inch in chord A-9-L about one week previous to August 27. This recital shows that the chords near the main pier both in the anchor arm and in the cantilever arm were under close observation for at least a week previous to the accident. These were the most heavily stressed compression members in the bridge.

We are satisfied that the structure was being closely watched and that had there been noticeable weakness at any points it would have been detected and recorded. There is no evidence of the existence of weak details except on the lower chord.

We therefore conclude both from the evidence of the witnesses, and from that of the wreckage, that the initial failure occurred in chords 9—R and 9—L, anchor arm.

Our opinion is that these two chords failed almost simultaneously by rupture of latticing or shearing of lattice rivets (see Appendix No. 17) and that the buckling of the ribs followed immediately. The cantilever arm commenced to drop, turning around the feet of the centre posts, and raising the anchor arm near the point of rupture. When the top of the centre post had leaned over perhaps 30 feet, (this estimate being made by the witness Chase) the centre post feet kicked off the pedestals, and both anchor and cantilever arms crashed down. The right truss of the anchor arm apparently fell faster than the left truss, for the top members of the arm have fallen towards the right, and the witness Delphis Lajeunesse noticed such a movement. The stub chords on the cantilever side which were attached to the centre post feet struck the coping of the masonry heavily, the marks of the contact on these chords indicating that the right post was falling the faster. The cantilever arm was controlled in its fall by the stiffness of the centre post, and by the resistance of the upper chord, and did not drop suddenly until the feet of the centre posts kicked off the pier. The centre post feet reached the ground first, carrying inwards before them the lower parts of panels 9 and 10 anchor arm; the remainder of the anchor arm was swung forward by the action of the upper chord in straightening out, under the pull of the cantilever arm, and moved around the top of the anchor pier as a fixed point. The damage to the lower chords from the fall was the more severe because the ends of the posts landed in the foundation pits dug for the falsework, and the chords themselves struck on the high ground between the pits. The forces that pushed the centre post feet out into their present position, as described by Mr. Kinloch, are a matter of conjecture; the holes dug by the feet in their fall are plain to view and are partly filled by sections 5 of the centre posts which are standing upright in them. As these sections are comparatively little injured and have not dug down into the ground, it is evident that they struck with but little force and that the ground was already shaped to receive them. The force of the fall was probably largely absorbed in the wrecking of sections 6 of the centre posts.

No description in words can give as correct an idea of the wreckage as will be obtained by a study of the photographs in Exhibit 34; the principal feature to be noted is the comparatively uninjured condition of all members except some of the lower chords, posts and sub-posts which by reason of their position had to bear the larger portion of the forces developed by the fall, and completely failed under them.

All connections except the splices of the lower chords proved to be as strong and in most cases much stronger than the body of some of the members they connected. The tension members, laterals and floor system call for little comment; the compression members and their splices have shown themselves to be the weakest parts of the structure.

The following is a statement of Mr. Walter J. Francis' observations of the wrecked structure, and which clearly describes certain phases of it:—

'The condition of the posts throughout may be said to be largely the result of the unyielding strength of the top chord eye-bar system, while the condition of the other members may be regarded as due to their fall to the earth and upon one another.

'Of more than 700 eye-bars in the wreck, only one has been found which has broken, and on all remaining ones there is not a sign of a crack or failure of any kind, notwithstanding the extreme punishment to which these members have been subjected. The broken bar, $1\frac{15}{16}$-inch x 15-inch, is undoubtedly the result of a heavy blow on the edge, about 18 inches from the centre of the pin. The bar parted about 4 feet from the centre of the pin, in acting as a beam. The fracture is fine grained, and although not of the highest class it would certainly be rated as good.

'Of about 60 pins in the accessible parts of the debris only one has any evidence of having been distorted. This pin is 12 inches in diameter, 8 feet 6 inches long, bored $2\frac{7}{8}$ inches diameter through its axis lengthwise. Its bend consists in having one end turned up about 5 inches, the curve being about $1\frac{1}{2}$ feet from this end. As this pin is at the joint where the eye-bar above referred to was broken, its condition is undoubtedly due to the same cause as that which broke the eye-bar.

'Speaking generally, the compression members throughout have suffered severely. They were generally composed of parallel laminated webs. In the maximum size of chords there were four vertical webs. Each web consists of four plates ranging from $\frac{13}{16}$-inch to $\frac{15}{16}$-inch in thickness, and one angle on each edge 8-inch x 6-inch x $\frac{15}{16}$-inch for outside webs, or 8-inch x $3\frac{1}{2}$-inch x $\frac{15}{16}$-inch for inside webs, the 8-inch leg being vertical. The finished width back to back of angles was 54 inches. The maximum length of these webs was about 57 feet. At each end the four webs are connected together top and bottom by cover plates varying from 6 feet to 10 feet long, the space between the cover plates being latticed with 4-inch x 3-inch x $\frac{3}{8}$-inch angles. The tower posts had four parallel webs, while in other posts there were two webs only, latticed for the greater part of their length with 3-inch x 3-inch x $\frac{3}{8}$-inch angles, set at about 60 degrees. Speaking generally, at and near the panel points of all these members, there were either internal diaphragms, or cover-plates, or both. Throughout the middle length of the members there were none. In the wreck the compression members are distorted in every conceivable manner, excepting at the panel points, where, as will be observed from the general photographs, the portions having internal diaphragms or outside covers are yet comparatively straight after enduring the forces of the fall. Between these stiffened portions the lattice work is torn, the laminated webs are parted, and the rivets sheared and pulled in every possible way.

'The component parts of the various built up members have been destroyed by all sorts of complications, as will be seen by reference to the accompanying photographs, to which descriptions are attached. These in themselves form an interesting study. In the selection of the 23 photographs attached hereto the intention was to choose only those which are typical of the general damage to the various pieces and those fractures which have been produced by simple and clearly defined forces. There are innumerable examples of destruction under extremely complicated sets of forces,

but these have no special scientific interest beyond the proof of the quality of the material.

'The evidence of heat produced by blows and friction is in many instances quite marked; one case was noted where the steel had been fused and drawn into shreds and small globules.

'Although quite secondary to the main members of the bridge, it is interesting to note that the 1⅝-inch anchor bolts holding the vertical steel in position on the anchor pier drew bodily out of the masonry. These bolts had the ordinary surface of a steel rod, were swedged on the opposite sides every 3-inch and were 4 feet 6 inches long. The holes in which they were set were drilled in the granite masonry just large enough to admit the bolt. They are said to have been grouted in with pure Portland cement. In every case where they received direct tension they pulled bodily out of the masonry.

'It is almost beyond comprehension that both the main pier and anchor pier should have withstood the shocks of the accident. There is no indication of any movement in, or general damage to, either of these piers. The arrises have been abraded in some cases, and where the main shoes left the pedestals the blow they administered to the coping and cornice moulding spawled the granite in one case for about 22 feet in length. The effect on the masonry, however, can only be absolutely determined by an exhaustive examination.'

As the lower chords call for particular attention we give here a memorandum of the condition of these chords after the accident. The other portions of the structure are sufficiently illustrated in the photographs and drawings already referred to.

This memorandum is part of Exhibit 54; it was prepared by Messrs. Cudworth, Kinloch & McLure, and was checked by the commissioners and found to be a correct description. It is as follows:—

A-1-L.

Starting with its pin connection with anchorage eye-bars, 79 ft. from C.L. of anchor pier, and about 35 ft. above the ground, A-1-L slopes at an angle of about 70 degrees to the horizontal, until it rests on the ground at a point 90 ft. from C. L. of anchor pier. Here the four ribs are broken entirely off, the west rib 3 ft. north of its splice to chord A-2-L, the west and east centre ribs at the field splice to A-2-L, and the east rib through the web at the south ends of splice plates. The top cover plate is still attached to A-2-L, and the bottom cover plate is torn off entirely. The latticing is still practically intact. (See photographs No. 10 and No. 18).

A-2-L

The portion of A-2-L separated from A-1-L, as above described, lies on the ground 96 ft. from C. L. of anchor pier, the break being about 6 ft. south of pin hole connecting hanger T-O-L. The chord bends to the east from this point to a point 118 ft. from C.L. of anchor pier, where all four ribs are twisted, and broken through the angles and web plates from the tops, half way down, (see photo. No. 18). At this point of break the deflection from a straight line is the max, and about 6 feet. From this break the chord dips downward at an angle of about 10 degrees with the horizontal, and slightly westward (see photo No. 17). The pin hole for connection of A-P-1-L is intact, and all four ribs of this chord are broken off at the field splice eight feet north of this pin hole. The top and bottom cover plates at the splice with A-3-L are torn from A-2 and fast on A-3. The latticing at point of break is broken, and all the remaining latticing badly bent up, but in position.

A-3-L

Starting with its splice with A-2-L, recorded above as broken, A-3-L has its four ribs in a straight line about parallel to axis of bridge, to a point 170 feet from C. L. of anchor pier, where the west rib is bent in toward the centre of chord, and the latticing broken, but rib itself uninjured.

The pin hole for connection of T-O-O hanger is intact, but all four ribs are broken through at the field splice eight feet north of this pin, right through the splice plates to A-4-L. At this point, the south end of A-4-L lies 4 feet above the north end of A-3-L, and 3 feet to the west (see photograph No. 16). At this splice between A-3-L and A-4-L, and bottom cover plate is torn from A-4-L and fast to A-3-L, and the top cover plate is torn from A-3-L and fast to A-4-L.

A-4-L

The four ribs of A-4-L run parallel to each other from their splice with A-3-L to a point 10 feet south of pin connecting post P-2-L, at which point the east rib spreads a foot toward the east till it reaches the pin hole of P-2-L post. At this point all four ribs are broken through. North of this pin hole, the two outer ribs are spread, but converge to their splice with A-5-L at which point the two centre ribs are broken off entirely, but the outside ribs, intact.

A-5-L

A-5-L runs continuously from its field splice with A-4-L to pin hole for connection of T-O-O-O-L hanger, where all four ribs are broken through. From this pin hole the chord runs straight to field splice. Here the three west ribs are broken off, but the east rib runs by the splice, 4 feet on to chord A-6-L where it is broken. The top cover plate at this splice is fastened at its east edge only, and the bottom cover plate torn loose from A-6-L and fast to A-5-L. The latticing has been little damaged.

A-6-L

A-6-L at its splice with A-5-L is offset about two feet towards the west, and from there runs in a straight line to the pin hole for connection of P-3-L. Here all four ribs are broken through. Beyond the pin hole the east rib is displaced slightly to the east to the field splice with A-7-L. At this splice the top cover plate is fast to east rib of A-6-L only, and bottom cover plate fast to four ribs of A-6-L only. The latticing is little damaged.

A-7-L

From its splice with A-6-L, A-7-L is deflected slightly to the west until it reaches the pin hole for connection of T-O-O-O-O-L hanger, where all four ribs are broken through. From this pin hole to the splice with A-8-L the ribs run straight. At the splice the two centre ribs are broken through the splice plates but the outside ribs are intact. The latticing is little damaged.

A-8-L

The ribs of A-8-L run straight from its splice with A-7-L for a distance of about 20 feet. At this point the west rib bends to the west about 90 degrees and rises in the air to a height of about 20 ft. The west and east centre ribs start to bend at the same point but come back again, forming a reverse curve, and burying themselves in a pile of scrap iron immediately beyond the pin hole for the connection for P-4-L. The east rib follows the same general direction, but its north end instead of turning downward, makes a hook toward the east. All four ribs are broken off at the pin hole for P-4-L, the piece from the west rib lying out on the beach about 25 feet from the present position of the west main pier shoe, and having attached to it two feet of the west rib of chord A-9-L, with the field splice intact. The latticing is almost entirely destroyed.

A-9-L

Beginning at the field splice with chord A-10-L, at which splice all four ribs are broken, the west rib of A-9-L runs south, at an angle of 45 degrees to the axis of the bridge towards the east to the pin hole for the connection of A-T-5-Z hanger, at which point it starts to bend eastward, turning through about 180 degrees in a length of 15 feet and thence running north eight feet. At this point it bends through 180 degrees again in a length of 10 ft., and then runs south and inclined upward at an angle of 40 degrees with the horizontal, to a point two feet beyond its field splice with A-8-L,

which splice is intact and fully riveted. At the last bend mentioned, three of the web plates are broken through.

Running parallel to west rib to T-5-Z pin hole, the west centre rib is there broken, but continues beyond, turning through 180 degrees, and running north for eight feet, then bending back 180 degrees, at which bend two of the web plates are broken through and running south to its former field splice with A-8-L, where it is broken.

The east centre rib runs parallel to west centre, but is not broken at pin hole, and at the last bend has only one web plate broken.

The east rib parallels the east centre rib through the first bend of 180 degrees to a point eight feet north of the pin hole, where it doubles over on itself and projects upward and toward the west to a height of 14 feet above the ground.

The distance from the field splice with A-8-L to the chain mark on west centre rib is 13 feet. The centre of max. bend of the chord is about 20 inches forward of this point, and the loose rivet discovered in the lattice angle is about midway between the chain mark and the centre of the bend. This bend lies about 15 feet south of the fracture in the floor beam between P-4 posts. All of the west end of A-9-L is still attached to T-5-Z hanger, and all of its four ribs are bent through 180 degrees at a distance of about 8 feet from the T-5-Z-L pin hole.

At the second bend mentioned in the east rib two web plates are broken through. The lattice angles are completely destroyed.

A-10-L

The four ribs of A-10-L, starting from its field splice with A-9-L, at which all four ribs are broken, runs in a straight line slightly inclined westward, with the ribs folded over and lying one on top of the other, the latticing being completely destroyed.

A-1-R

Starting with its connection with the anchorage eye-bars, A-1-R dips downward at an angle of about 70 degrees to the horizontal. At a point 6 feet distant horizontally from its south end it is crippled through all four ribs, and bends toward the east, the flange angles being cracked through here and the latticing torn off. Turning again 90 degrees it runs straight down into the ground at the pin hole for the connection of T-O-R hanger, at a very short distance beyond which, buried in the mud, the four ribs are broken off through the webs. The field splice 4 feet south of A-T-O-R hanger pin hole is intact on the two outer ribs, but slightly loosened up on the inner. The top cover plate is on, but the bottom one partly torn off.

A-2-R

Beginning at the break mentioned as north of the T-O-R hanger connection, this end of A-2-R has been thrown westward to a position 138 feet from C-L of anchor pier and 5 feet west of original east truss line, the chord turned up on its west side, and running northeast to a point 155 feet from C L of anchor pier and 31 feet east of original line of east truss. The chord has a long bend at its centre, and the latticing is badly bent up, but for the most part still fast to the chord. All four ribs are broken completely through just south of the P-1-R pin hole, and form the end last located. The remainder of the chord lies at the foot of P-1-R post and runs north from that to its field splice with A-3-R, at which the east rib is broken three feet north of splice on chord A-3-R, and the other ribs broken right at the splice.

A-3-R

At a point six feet from its field splice with A-2-R this chord bends sharply to the east for five feet and then back again to a direction about parallel to axis of bridge. At the pin hole for connection of A-T-O-O-R hanger, the east rib only is broken. At the field splice with A-4-R the east rib is intact and the other three ribs broken through. The bottom cover plate is fast to east rib of both chords, and top cover plate

to all ribs of A-4-R and to east rib of A-3-R. The latticing is in good condition at south end and broken off at north end.

A-4-R

From its field splice with A-3-R, A-4-R runs straight to a point 10 feet south of pin hole for connection of P-2-R post, where the outside ribs bulge out around the pin hole to the field splice with A-5-R. At the pin hole all four ribs are broken through. At the splice the west rib is partly and the other three entirely broken through. The batten plate on chord just south of P-2-R is entirely destroyed. The latticing is little damaged.

A-5-R.

Runs straight from its field splice at south end to the pin hole for connection of T-O-O-O-R hanger, where all ribs are broken through. From pin hole to field splice at north end the chord is tipped up in the air at an angle of 45 degrees to the horizontal. At the field splice the splice plates are stripped off the two outside ribs. On the inner ribs the splice plates are broken through. Latticing partly broken.

A-6-R

Runs straight from its field splice with A-5-R to pin hole for connection of P-3-R post, where the four ribs are broken through. From the pin hole to its field splice with A-7-R chord inclines slightly west. At this splice all four ribs are broken, and the short section thus left is tipped up about 15 degrees with the horizontal. The latticing has been little damaged.

A-7-R

Starting at an offset of 18 inches east from A-6-R at splice, A-7-R runs straight to pin hole for connection of A-T-O-O-O-O-R hanger, where the four ribs are broken through at pin hole. From this point, t ofield splice with A-8-R the chord inclines slightly westward. At the latter splice everything is intact except the bottom cover plate which is partly broken off from east rib. Latticing bent up, but not badly broken.

A-8-R

Running six feet north from its field splice with A-7-R the chord is straight. At this point the three west ribs take a sharp bend through almost 90 degrees to the east for six feet, followed a little further north by a similar but wider bend in east rib, all four ribs turning north again to meet the pin hole for connection of A-P-4-R post, at which point the ribs are all broken off.

The west rib runs from this pin hole to splice which is intact, and continuing on to the west rib of A-9-R makes a sharp bend of 180 degrees to the west and south, and in a few feet, again turns about 75 degrees to the west and is broken off through its web about opposite the pin in foot of A-P-4-R.

The west centre rib parallels the west rib, across the field splice, continuing on to the same rib in chord A-9-R and terminating in a broken end at about the same point as the west rib.

The east centre rib runs from the pin hole to the field splice, and is there broken off.

The east rib runs from the pin hole across the splice which is intact, on to the same rib of chord A-9-R, turning to the east and south, through about 150 degrees, and terminating in a broken end at a point about two feet north of the pin at foot of post A-P-4-R.

A-9-R

Starting at the field splice with A-10-R this chord runs south, almost directly underneath chord A-7-R, to the pin hole for connection of A-T-5-Z hanger, at which point all four ribs are broken. From here the four ribs turn to the west about

90 degrees and run completely under A-7-R. After passing under the latter chord, the east rib continues almost directly westward for a distance of 20 ft. and terminates in a broken and twisted end, which probably matches the other end of this rib described under chord A-8-R, and located about 75 ft. distant.

After emerging from underneath chord A-7-R the other three ribs continue the 90 degree bend to one of about 180 degrees, and run directly north, the east and east centre ribs terminating in ends broken and twisted, directly opposite and just west of field splice between A-9-R and A-10-R, and the west centre rib continuing its course north to its faced end, opposite and directly east of field splice between A-7-R and A-8-R, and before reaching there, having three of its four plates torn from it and doubled back, and the fourth broken half through, and twisted completely around. In this neighbourhood there are numerous small pieces of plates and angles that can readily be identified as having once belonged to chord A-9-R. Latticing on this chord is completely destroyed.

A-10-R

The field splice between A-9-R and A-10-R is partly broken. Starting from that point, A-10-R runs north, and inclining slightly eastward to a point near the south end of the stub chord A-11-R, pinned on the 24-inch pin, its field splice with which is entirely broken. The ribs of A-10-R are comparatively straight, but are piled over, one on the other, and the latticing entirely destroyed.

A-11-R and L.

These V-shaped stub chords are still in the positions originally placed, on the 24-inch pins holding them to the main pier shoes. Their field splices with both the number 10 chords of anchor and cantilever arms have been broken off entirely, but the chords themselves damaged but little.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY.
J. GALBRAITH.

APPENDIX No. 13.

AN EXAMINATION OF THE VARIOUS FULL-SIZED COLUMN TESTS THAT HAVE BEEN MADE IN AMERICA, ACCOMPANIED BY DIAGRAMS SHOWING THE RESULTS OF THESE TESTS.

In view of the circumstances accompanying the accident of August 29th, it was necessary for us to investigate the design of the lower chords and the data that were at the disposal of the designer (The Phœnix Bridge Company's engineer) when he began his work. This investigation was commenced by an examination of all obtainable records of column tests.

The column formulas used in practice are, broadly speaking, empirical formulas, framed to suit the results of these tests.

In examining the records, a process of elimination was adopted, the object being to select those tests which most nearly corresponded to the Quebec Bridge conditions. The following are the considerations upon which the selection was made.

(1) No tests on solid sections were used, because the bridge chords were built-up members and apparently failed from weakness of connecting details, the conditions being absolutely different from those existing in a solid section.

(2) No tests on sections that have been proved defective were used, except that certain of the Buchanan tests, which were published in 1907, have been selected on account of the large sections of the test pieces, although these were not of the most approved design.

(3) No tests on members whose failure was caused by defects in the testing apparatus were used. In the earlier tests special ends were fitted to many of the test columns with unsatisfactory results.

(4) It was intended to exclude all tests on members having less than 10 square inches area, but some tests on sections having areas between 7½ and 10 square inches have been included.

(5) When the ratio $\frac{l}{r}$ for any column exceeded about 120 the test results were not used.

The records consulted were:—

(1) J. M. Moncrieff (Am. Soc. C. E., Vol. XLV., 1901.)

This paper, which was written by an English engineer, contains perhaps the most complete compilation of column test data that has ever been published. It was consulted for reference to original authorities. The records contained in it show that there were practically no English or European tests that would not be excluded by the fourth consideration above mentioned.

(2) 'Tests of Metals.'

This is the official record of all tests made at the United States Arsenal at Watertown, Mass. The complete file of these volumes, publications of which began about 1881, was examined. No tests of interest were found in any volumes issued after 1884. The results from tests on wrought iron columns of the Phœnix box and latticed channel types have been selected. The results of 99 of these tests have been used. The specimens varied in cross section from 7 square inches to 22 square inches, there being 6 with areas between 20 and 22 square inches and 14 with areas between 15 and 20 square inches.

(3) G. Bouscaren (Am. Soc. C. E., Volume IX., 1880.)

The tests recorded in this paper were made between 1875 and 1879 in connection with the building of the Cinninnati Southern Railway. They included tests on wrought iron columns of the box and latticed channel types. In all 9 tests were selected for use. This series of tests has possibly had more influence upon the detail of bridge design than any other series that has been made, as the rejection of various types of columns and the adoption of various modifications in detailing directly resulted from it. The small number of tests that have been selected for this record shows how greatly the tests were needed at the time. The cross section varied from a minimum of about 11 square inches to a maximum of about 14 square inches, with the exception of one box column which had an area of 26.05 square inches. The metal used developed an ultimate strength of between 52,000 and 55,000 lbs. per square inch, Mr. Bouscaren's specification of 1875 calling for an ultimate strength of 60,000 lbs. per square inch in tension.

(4) Clarke, Reeves and Company (Am. Soc. C. E. Volume XI, 1882).

This firm, which was the predecessor of the Phœnix Bridge Company, published in this paper the results of a series of tests on Phœnix columns which were made for them in 1879 and 1880 at the Watertown Arsenal, the material being wrought iron. There were 22 tests in all.

It was found necessary to alter the 'breaking load' on some of the shorter columns given in the records, as an examination of the diary of the tests showed that real failure had occurred considerably before the metal managed to escape from the following up of the machine.

Clarke, Reeves and Company's specification of 1871 calls for iron of an ultimate strength of from 55,000 to 60,000 lbs. per square inch. Twenty of the columns had

a sectional area of about 12 square inches and two a sectional area of 18.3 square inches.

(5) C. L. Strobel (Am. Soc. C. E., Volume XVIII, 1888).

The tests recorded in this paper were made in 1887 upon columns of H-shape built up out of 4 Z-bars with a latticed web. The material was wrought iron, and the results of nine tests have been used. The sectional area in each case was between 9 and 10 inches.

(6) J. C. Dagron (Am. Soc. C. E., Volume XX, 1889).

This series of eight tests, all of which have been used, were made in 1884-5. The columns were of the latticed two-channel type, the channels being built up. The material was high steel, the ultimate strength being given at 84,000 lbs. per square inch and the elastic limit at 53,000 lbs. per square inch. The columns were between 8 and 14 square inch cross sections.

(7) Professor W. H. Burr. 'The Elasticity and Resistance of the Materials of Engineering.'

In this book there is given a full resumé of column test data, including 4 tests on Phœnix columns made in 1873, the results of which have been used. The columns were between 8 and 14 square inches in cross section.

(8) C. P. Buchanan (Engineering News, December 26, 1907).

In this paper are given the results of 19 tests made between 1888 and 1900, the sections of the specimens varying from about 14 square inches to 33 square inches, these being the largest columns that had been tested previous to the investigations made in connection with our inquiry. The results were not made public until the date above mentioned and were not available for use of the Quebec bridge designers. Twelve of the specimens were of wrought iron, three of Bessemer steel and four of open hearth steel, these last four being of the grade known as 'structural steel,' which is at present in general use for bridge work. Only six of the specimens were strictly symmetrical. The columns were of the 'H' two-channel and upper chord types. All the results have been used, although on account of unsymmetrical sections and eccentric loading in several cases, high ultimate strength was not to be expected.

(9) J. A. L. Waddell (Engineering News, January 16, 1908).

This paper gives the results of six tests upon structural steel columns of the two-channel type. The tests were made about 1907. All of the results have been used, the column sections being 17·44 square inches in area. The results of the tests on nickel steel columns which were made at the same time have not been included.

The results of 176 tests in all have been plotted, the cross section of the largest column being less than 33 square inches in area, and that of the smallest greater than 7½ square inches; three columns had cross sections greater than 30 square inches, 9 greater than 25 square inches, 16 greater than 20 square inches and 20 greater than 15 square inches. The results of the tests are plotted in drawing No. 20, and are divided into three groups, viz.: flat-ended wrought iron columns, pin-ended wrought iron columns and pin-ended steel columns.

The following conclusions may be drawn from this study:—

(1) Very few tests have been made on full-sized steel columns, and some of those that have been made are upon unusual grades of material.

(2) The experiments upon which modern practice is largely depending were made at least twenty years ago, and upon a grade of material which is not now in use in bridge construction.

(3) The decrease of strength with increase of the ratio of $\frac{l}{r}$ is, in the case of flat-ended wrought iron columns, not clearly discernible on the diagram in drawing No. 20.

(4) This decrease is discernible on the diagram in the case of pin-ended wrought iron columns, but it is not nearly so rapid as the decrease indicated by the column formula adopted in the amended specifications for the Quebec bridge.

(5) It is evident from the particulars of many tests that the size and strength of the pin used have an appreciable effect on the results obtained, but the amount of this effect has not been determined.

(6) The relation between the strength of a column as determined by test and as calculated by formula varies greatly.

(7) No series of tests have been made to determine the relative stresses in the various parts of a built-up column.

(8) The strength of a column is greatly affected by what have been considered minor features in the end details.

(9) A compression member of usual design and dimensions cannot be expected to develop an ultimate strength much greater than about one-half of the ultimate strength of a tension member made from the same material.

(10) No tests have been made on columns of the form of the Quebec lower chords nor on any having more than about $\frac{1}{25}$ of the cross section of these chords.

That the results of laboratory tests should not be rigidly followed in field practice is axiomatic, but the extent to which they can be safely accepted is a matter of judgment. During the last 25 years, a failure similar to that of the Quebec bridge has been, we believe, unknown, and as compression members designed in accordance with the results of the Bouscaren tests have been uniformly successful, little doubt existed in the minds of practising bridge engineers concerning them.

There is no definite evidence to show that either Mr. Cooper or Mr. Szlapka ordered any investigation to be made of the tests data that were available, and when the comparative magnitude of the undertaking is remembered, it is difficult to explain their failure to check their conclusions on the Phœnix testing machine, which was at their disposal.

On the drawing the results of the tests are shown, arranged according to the ratio $\frac{l}{r}$ for each column. The form of the section upon which each test was made,—double channel H, box Phœnix, or upper chord—is indicated by miniature sections.

It should be remembered that, previous to the Quebec disaster, the insufficiency of the existing knowledge of column action had been widely recognized, and programmes for additional testing were under consideration both by the American Society for Testing Materials and by an independent committee of prominent engineers, acting in co-operation with the officers attached to the United States Arsenal at Watertown. It is generally felt that modern bridge work has grown to such dimensions that further investigations are desirable.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY.
J. GALBRAITH.

APPENDIX No. 14.

A COMPARISON OF THE STRESSES IN THE SEVERAL MEMBERS OF THE MAIN TRUSSES, COMPUTED FROM THE BRIDGE AS FINALLY DESIGNED, WITH THE STRESSES AUTHORIZED BY THE SPECIFICATIONS.

The nineteen tables acompanying the report of Mr. C. C. Schneider, consulting engineer, upon the design of the Quebec bridge, are self-explanatory. All Mr. Schneider's results have been compared with corresponding figures furnished by the Phœnix Bridge Company and in general are found to slightly underrun them. They show that the calculations of the Phœnix Bridge Company were carefully and accurately made. (*See* Exhibits Nos. 102 and 108.)

Drawing No. 4 has been prepared for the Commission and revised by the Phœnix Bridge Company. This drawing shows the maximum stresses arising from dead load, plus 1½ live load plus ½ wind, this loading having been used to some extent in the original calculations at Mr. Cooper's direction. The only difference in the calculations leading up to the two sets of figures on the drawing lies in the dead load used; for the first set the dead load assumed in the designing was taken, and for the second, the actual dead load obtained from the built members. It will be noted that the error of stresses in the main chords near the centre posts, due to this error of assumed dead load is fully 10 per cent.

No satisfactory exploration of the occurrence of this error has been offered. On minor bridges, with a given live loading the weight of metal is known not to vary greatly with details of design and in some offices revision of the assumed dead loads for such bridge is not the rule ; but no information from which to predict the weight of the Quebec bridge existed, and the probability of a serious mistake in the first estimate for weight would be apparent to a cautious designer.

The fact is that Messrs. Deans, Szlapka and Cooper permitted the shops and rolling mills to commence work without taking any steps to test the correctness of the assumed dead load, and the probable dead load does not appear to have been estimated from the plans until at least eighteen months after the work of fabrication was commenced. (See Appendix No. 8.)

A list showing the dates on which each shop drawing was computed is filed as Exhibit No. 125, and it will be noted that the work of désigning was so far advanced by the beginning of 1905, that the preliminary estimates of dead load might then have been revised with considerable accuracy. By reference to Appendix No. 8, it will be seen that the percentage of error in the original estimates for all parts of the spans was roughly the same.

We are of opinion that no manufacturing should have been done until the designers had so far advanced with their work as to be able to make a proper estimate of the weight of the bridge. (See clause 3 of 1898 specification Exhibit No. 21). Before completing the drawings for use in the shop the weight of the various parts should have been computed as a check on the estimated weight of the bridge. As a matter of fact this procedure was not adopted and manufacturing was commenced in July, 1904, without any such checking, although the specifications called for it, and the contract practically demanded it. (See Appendix No. 8).

The designing office had accumulated sufficient information to enable it to make a close estimate of the weight of the bridge but did not do so. On the contrary, work continued as if their assumptions had been correct.

That Mr. Cooper fully intended to permit stresses in excess of 24,000 lbs. per square inch under the conditions used for drawing No. 4 is shown by the following letters:—

August 6, 1904.

PHOENIX BRIDGE COMPANY,
Phoenixville, Pa.

MY DEAR MR. SZLAPKA,—I have tested the proportions of the members of the anchor arm under the following maximum loading for my personal satisfaction—viz.: Dead plus 1.5 live plus 25 lbs. of wind ($\frac{1}{2}$ of your wind strain) and find that the only members exceeding 24,000 in tension or $24,000-100\frac{L}{R}$ for compression are—

The lower chord which has 26,500 and is *all right* and Towers L which should have 108 square inches.

Towers B which should have 99 square inches to come within the above conditions.

This is such a slight matter I request for the sentiment of the thing that you change those last two members to the above sections if it does not inconvenience anything.

Yours very truly,

THEODORE COOPER.

August 9, 1904.

THEO. COOPER,
Consulting Engineer,
New York, N.Y.

DEAR SIR,—I have your kind letter of August 6 in reference to increase of section of members 'T L O O O O O and T B O O O O O' for combination of stresses due to dead load plus $1\frac{1}{2}$ live-load plus wind.

I will gladly comply with your request and will also apply the same combination to all other members to satisfy myself that the unit stresses are in proportion not higher than those on the two above mentioned members.

Yours respectfully,

P. L. SZLAPKA.

The propriety of the selected stresses is discussed in Appendix No. 18.

HENRY HOLGATE,
Chairman.

J. G. G. KERRY.
J. GALBRAITH.

APPENDIX No. 15.

A DESCRIPTION OF THE VARIOUS EXPERIMENTAL RESEARCHES THAT HAVE BEEN MADE IN CONNECTION WITH THE BUILDING OF THE QUEBEC BRIDGE AND DURING THIS INQUIRY.

The Phœnix Iron Company possesses the most powerful machine for compression tests in existence; unfortunately, some doubt exists as to the accuracy of the records obtained from it. As a result of a series of tests made in 1897, the New York Department of Buildings places its error in compression at 15 per cent in excess; in tension,

however, its results seem to be in agreement with those obtained with other machines on similar material. In spite of these doubts, however, this machine has been of invaluable service to the engineering profession.

In the evidence, Mr. David Reeves, the president of the Phœnix Bridge Company, states that he had given orders 'that all the special tests advised by the consulting engineer, Mr. Cooper, or by our own engineers, arising from the unusual size of the bridge, be promptly and fully made,' so that from the outset the designers of the Quebec bridge had at their disposal both the equipment for making tests and the authority to use that equipment.

The evidence shows that along certain lines these facilities were by no means neglected, and we are of opinion that had Mr. Cooper and Mr. Szlapka realized how limited is our knowledge of the strength of compression members, they would have made as much use of the testing machine for compression tests as they did in connection with the eye-bars.

The appliances and tackle that were so successfully used in the erection were tested when necessary, and the only failure of which we have record was in the hook that was lifting A—9—L in the Chaudière yard.

Some of the tests which were made are as follows:—Two plates about 28″ x 2⅛″ in section were tested in tension (see Exhibit 85), to determine the efficiency of the connection between the two pins to be used at certain intersections. The plates were tested with 12″ pins and reinforced bearings; the records filed are rather meagre. One plate dished at one pin bearing when the stress amounted to 35,200 lbs. per square inch, the test having continued without sign of failure to 26,000 lbs. per square inch. In the test of the second plate the rivets began to work loose at a stress of 16,000 lbs. per square inch; the test was discontinued before failure, when a stress of 26,000 lbs. was reached.

An eye-bar, 16″ x 1¾″, was made into two by cutting and reheading. One half was bent into a long 'S,' the maximum deviations from the line between centres of pin holes being about 3½″ and 4¾″; the length centre to centre of pin holes was about 17 feet. The other half was tested to destruction as a straight bar and failed under a stress of 57,990 lbs. per square inch; 14-inch pins were used. The bent half stood a stress of 61,340 lbs. per square inch before it failed. The bends were made in the plane perpendicular to the pins. The test was assumed to indicate the negligible effect of waves and bends in tension members.

Mr. Cooper having questioned the efficiency of the device for adjusting the position of the suspended span on account of the friction between the pins and the toggle eye-bars, tests were made to determine the correctness of his opinion. The tests were not conclusive, and Mr. Cooper decided that some entirely different device should be used at the north end of the suspended span.

An important series of tests was made at Mr. Cooper's direction upon the deformation of eye-bars under strain. The usual record of tests upon full-sized eye-bars will be found in Exhibit No. 86; 73 tests in all were made. Squares were scribed on the heads of a number of these, and observations were made both of the flow of the metal near the eye and of the deformation at the pin-bearing. This study has been fully described by Mr. Cooper in his paper entitled 'New facts about eye-bars,' presented at the meeting of the American Society of Civil Engineers, March 21, 1906. The shapes of the eye-bar heads after the tests are fully shown in Exhibit 104.

Alterations were made in the dimensions of the eye-bar heads as a result of these tests and the set at the pin-bearing was allowed for in the camber diagrams. The above were all of the special tests made in connection with the design of the bridge.

After the collapse of the bridge the Phœnix Bridge Company, at its own cost and on its own initiative, built and tested the chord shown on Drawing No. 22. This model chord had, as far as possible, the same relative dimensions as the No. 9 chords

of the Quebec bridge and yet was small enough to be broken in the Phœnix Iron Company's testing machine. The test was made on November 21 and 22, 1907, and was under the general direction of Professor W. H. Burr. By the courtesy of the Phœnix Bridge Company we are able to give the text of Professor Burr's report:—

New York, December 23, 1907.

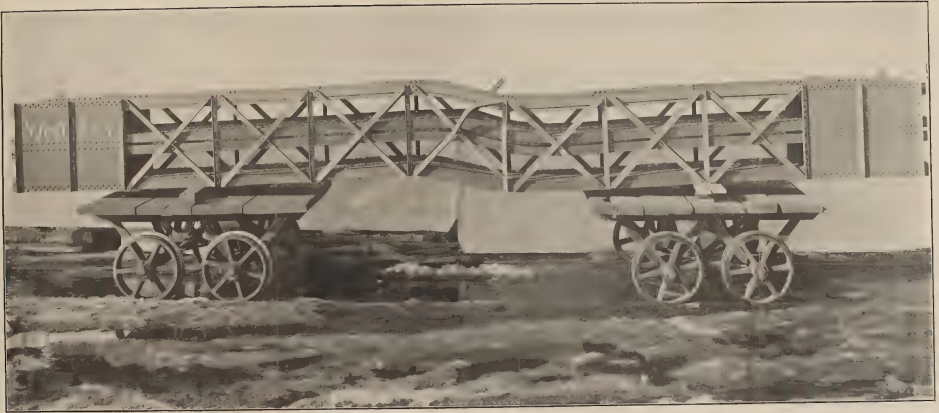
Mr. David Reeves, President,
The Phœnix Bridge Company,
Philadelphia, Pa.

Dear Sir,—In accordance with your instructions a model chord section was built to a linear scale of one-third of the lower chord section 9 of the anchor arm truss of the Quebec Bridge and was tested to destruction, under my direction and supervision, at the shops of the Phœnix Bridge Company at Phœnixville on November 21st and 22nd of the current year. The purpose of this test was to secure all possible information regarding the circumstances and method or other features of the failure of that chord which could be disclosed by the test of the model column in question.

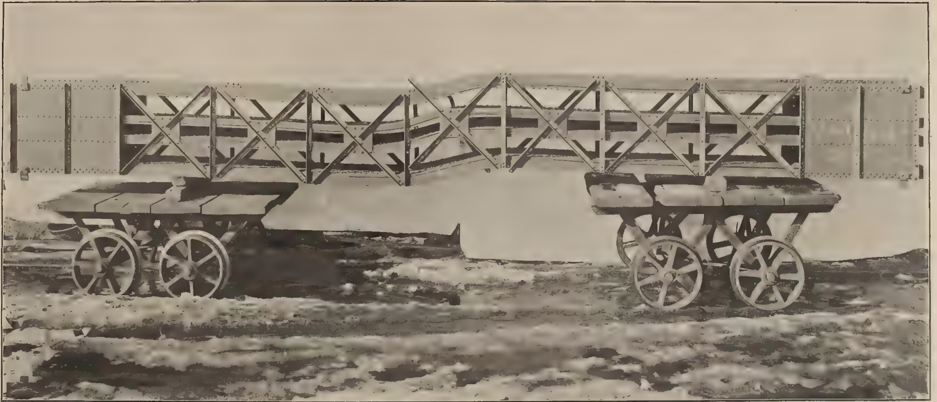
This chord section was built of four ribs 54 inches deep, with 4 in. x 3 in. x $\frac{3}{8}$ in. double angle latticing. Its area of cross section was 780 sq. in.

All the linear dimensions of the model were exactly one-third of those of the full size chord section, making the area of cross section (86.526 sq. ins.) one-ninth of that of the full size member and the volume of metal, with the exception about to be noted, one-twenty-seventh of the original member. This exception arises from the fact that the actual chord member as built, 57 ft. $\frac{9}{32}$ in. in length, had a heavy chord joint in it 10 ft. 6 in., a little more than twice the depth, from one end. Furthermore, the full size chords were bored for 12-inch pins, and pins of the same diameter were used for the end bearings of the model chord section. It is manifestly impossible to reproduce in a test precisely the conditions existing in the structure at the time of its failure, but it is believed that the end conditions employed in the test and the accurate reproduction by scale of the main dimensions and nearly all the dimensions of the details in the model enable the nearest approach to the actual conditions of the structure to be secured. It is believed that these unavoidable and subordinate departures from the actual conditions of the chord member did not sensibly affect the conditions of failure in the testing machine or the ultimate load carried by the model.

The blue print plans accompanying this report show both the working drawings of the original chord members 8 and 9, including the joint mentioned above, and those of the model chord precisely as it was built as well as in its condition after test, the latter plan having been made from accurate measurements of the failed member immediately after its removal from the testing machine. The blue prints of the model show the four webs of the original chord accurately reproduced by scale, making the depth $18\frac{3}{16}$ in. and the length 19 ft. As the plans of both the actual chord and the model show every main and detailed dimension it is not necessary to repeat them here. It should be stated, however, that each of the two interior ribs were composed of one 18-inch x $\frac{5}{16}$-inch plate, one 18-inch x $\frac{1}{4}$-inch plate, two $15\frac{5}{16}$-inch x $\frac{5}{16}$-inch side plates, and two $2\frac{11}{16}$-inch x $1\frac{1}{4}$-inch x $\frac{5}{16}$-inch angles; and that the two exterior ribs were each composed of one 18-inch x $\frac{5}{16}$-inch plate, two 18-inch x $\frac{1}{4}$-inch plates, one $12\frac{5}{8}$-inch x $\frac{5}{16}$-inch side plate, and two $2\frac{11}{16}$-inch x 2-inch x $\frac{5}{16}$-inch angles. The latticing was a double oblique system of $1\frac{11}{32}$-inch x 1-inch x $\frac{1}{8}$-inch angles, with $1\frac{3}{16}$-inch x 1-inch x $\frac{1}{8}$-inch crossing angles at the panel points of the former at right angles to the axis of the member. All of these lattice angles had two $\frac{7}{24}$-inch rivets at the ends of each with a single rivet at each crossing of the interior flange angles of rims, as clearly shown on the plans. The linear scale of one-third of the actual dimensions required the rivets used to be $\frac{1}{4}$-inch, $\frac{5}{16}$-inch and $\frac{7}{24}$-inch in diameter, also as shown on the plans, the $\frac{7}{24}$-inch rivets being turned down from an original diameter of $\frac{5}{16}$ of an inch. Similarly the $2\frac{11}{16}$-inch x 2-inch and the $2\frac{11}{16}$-inch

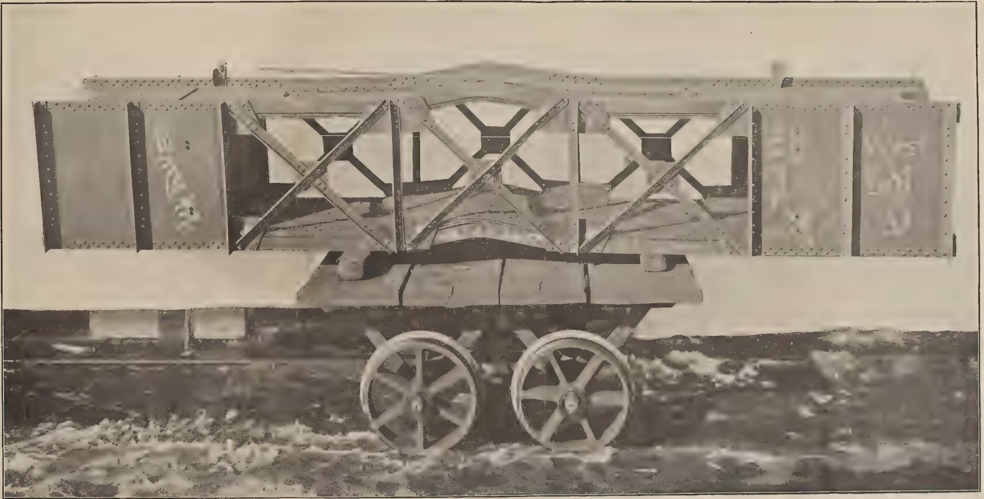


PHOTOGRAPH No. 1 REFERRED TO IN APPENDIX No. 15, VOL. I.

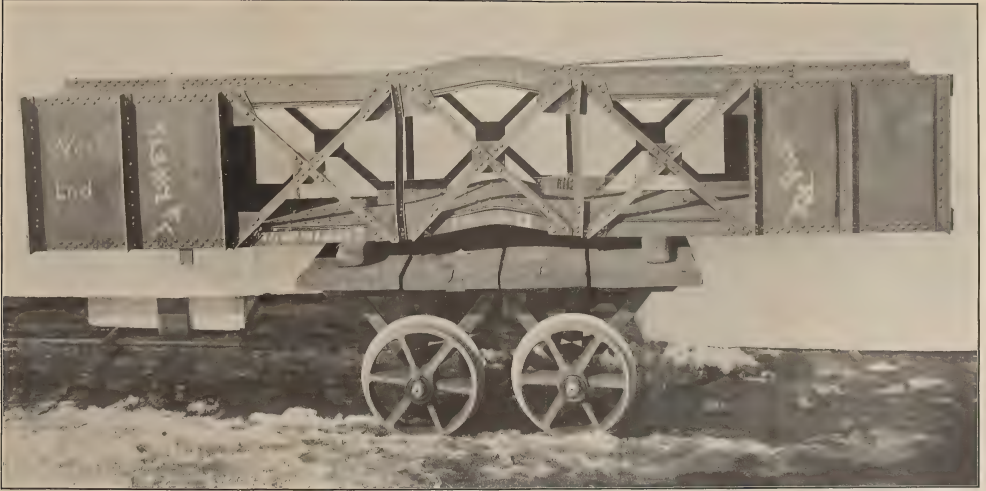


151—vol. i—1908

PHOTOGRAPH No. 2 REFERRED TO IN APPENDIX No. 15, VOL. I



PHOTOGRAPH No. 3 REFERRED TO IN APPENDIX No. 15, VOL. I



PHOTOGRAPH No. 4 REFERRED TO IN APPENDIX No. 15, VOL. I

x $1\frac{1}{4}$-inch rib flange angles were planed down from 3-inch x 2-inch x $\frac{5}{16}$-inch pieces. The lacing angles were also planed from $1\frac{1}{2}$-inch x $1\frac{1}{2}$-inch x $\frac{1}{8}$-inch and $\frac{5}{32}$-inch angle to the dimensions given above. All rivet holes were drilled.

The method of construction of the model was such as to leave it in true and accurate condition. The web plates were laid off by wood templet, except the pin plate holes, and drilled while the pin plates were drilled from iron templates. The pin plates were then used as templates for the drilled holes at the ends of the web plates. One web plate for each rib thus drilled was used as a drilling template for the other plates of the same rib, the blank plates being bolted to the drilled plate for this purpose. In the same manner the blank flange angles were bolted to the drill webs and drilled from the latter as a templat. Rivet holes required for lattice angles were drilled from iron templets, but the batten plates first drilled were used for drilling templets after the chord was completely assembled. After the component parts of the ribs were drilled they were taken apart, cleaned, painted and bolted together for riveting. The latter was done both in web and lattice angles with pneumatic hammers. The lattice bars were drilled like the other parts of the model. After the riveting was completed the pin holes were bored and subsequently the ends were faced to proper dimensions in a rotary planer. All the metal used for the main parts of the model column was medium steel, but soft steel was used for rivets. The steel plates were furnished by the Lukens Iron and Steel Company, of Coatesville, Pennsylvania; but the angles were supplied and rolled by the Phœnix Iron Company, of Phœnixville, Pennsylvania. The rivets were purchased in Philadelphia.

In order that the character of the metal employed might be completely determined, tensile tests were made of both plates and angles and shearing tests of both the $\frac{7}{24}$-inch rivets used in the model and $\frac{7}{8}$-inch rivets used in the full size chord.

The following tabular statements show the results of all these tests and of representative specimen tests of the metal used in the chord member 9 as actually built, together with chemical analyses exhibiting the main elements of interest in such structural material:—

TENSILE TESTS OF PLATES AND ANGLES.

$1\frac{1}{2}$-IN. x $1\frac{1}{2}$-IN. x $\frac{1}{8}$-IN. ANGLES.

Date.	Heat No. or Size.	Pounds Per Sq. In.		Per Cent.		Fracture.
		Elas. Lim.	Ultimate.	Strength in 8-in.	Reduction.	
Nov. 6..........	1402	52,520	65,660	27·0	55·6	Silky.
" 6............	1402	50,000	63,460	23·0	57·7	"
" 6............	1402	51,900	62,500	27·5	59·6	"
" 6	1402	50,340	61,300	21·0	54·4	" $\frac{1}{2}$ cup.
" 6...........	1402	50,360	65,700	20·5	48·2	" "

3-IN. x 2-IN. x $\frac{5}{16}$-IN. ANGLES.

Date.	Heat No. or Size.	Elas. Lim.	Ultimate.	Strength in 8-in.	Reduction.	Fracture.
Nov. 5..........	1,402	42,300	63,040	31·25	61·3	Silky.
" 6...........	1,402	41,780	62,100	32·0	54·0	"

$\frac{5}{16}$-IN. PLATES: TEST SPECIMENS 1·045 INCH WIDE.

Date.	Heat No. or Size.	Elas. Lim.	Ultimate.	Strength in 8-in.	Reduction.	Fracture.
Oct. 29............	13673	38,270	65,420	29·0	53·1	
" 29..........	13676	37,350	64,200	30·0	53 1	

PLATES OF CHORD MEMBER 9 AS BUILT.

Nov. 1	$54\frac{1}{2}$ x $\frac{7}{8}$	38,840	60,680	26·5	53·0	Silky cup.
" 1	$54\frac{1}{2}$ x $\frac{3}{16}$	40,810	61,440	25·5	51·4	" ang.
" 1	$54\frac{1}{2}$ x $\frac{3}{16}$	42,000	67,700	23·0	50·5	" "
" 1	$54\frac{1}{2}$ x $\frac{3}{16}$	40,780	65,540	24·5	49·0	" cup.

ANGLES OF CHORD MEMBER AS BUILT.

Sept. 14	8 x $3\frac{1}{2}$ x $\frac{15}{16}$	38,000	61,900	27·0	52·6	Silky cup.
" 14	8 x 3 x $\frac{15}{16}$	37,120	63,920	29·0	50·6	" "
" 14	8 x 6 x $\frac{15}{16}$	39,460	62,300	30·0	47·1	" "
" 14	8 x 6 x $\frac{15}{16}$	38,890	61,300	32·5	49·2	" "
" 30	4 x 3 x $\frac{3}{8}$	41,730	67,640	29·5	0·26	" ang.
Nov. 18	4 x 3 x $\frac{3}{8}$	42,710	64,860	27·0	0·27	" "

SHEAR TESTS OF RIVETS, NOV., 1907.

Size of Rivets.	Ultimate resist. in lbs.		Per square inch.	Average.
$\frac{7}{24}$-in. diameter	59·700	58·200	59 700	59·200
$\frac{7}{8}$-in. "	50·420	50·875	51·380	50·960

CHEMICAL ANALYSES.

—	Car.	Phos.	Man.	Sul.
$1\frac{1}{2}$-in. x $1\frac{1}{2}$-in. x $\frac{1}{8}$-in. angles	·16	·038	·51	·037
$\frac{5}{16}$-in. plates	·21	·016	·40	·023
" "	·23	·025	·42	·024
$54\frac{1}{2}$ x $\frac{7}{8}$ plates	·17	·01	·46	
$54\frac{1}{2}$ x $\frac{13}{16}$ plates	·17	·01	·46	
" "	·26	·007	·34	
" "	·26	·007	·34	
8 x $3\frac{1}{2}$ x $\frac{15}{16}$ angles	·16	·041	·36	
" "	·16	·041	·36	
8 x 6 x $\frac{15}{16}$ "	·17	·052	·39	
" "	·17	·052	·39	
4 x 3 x $\frac{3}{8}$ "	·18	·036	·66	
" "	·19	·05	·41	

The specimen tests of the plates and angles used in the actual chords were selected by me out of a large number so as to give a reasonable and comprehensive view of all and they are fairly representative. It will be observed that the usual effect of rolling thin metal necessarily finishing at a lower temperature than that in the heavier sections is apparent in the high elastic limits of $1\frac{1}{2}$-inch x $1\frac{1}{2}$-inch x $\frac{1}{8}$-inch angles. The same effect, but to a small degree only, is probably discoverable in the $\frac{5}{16}$-inch and $\frac{3}{8}$-inch angles. This marked effect in the lattice angles of the model column has a distinct bearing upon the final results of the test. A similar general observation, and to a marked degree, applies to the higher unit shearing values of the $\frac{7}{24}$-inch rivets as compared with those of the $\frac{7}{8}$-inch rivets of the full size member.

After placing the model column in the machine and under a load producing a stress of 12,000 lbs. per square inch it was thought that a buckling or bulging of the web plates was discovered to the extent of ·034 inch near the west end of the north rib, but this was found not to increase under further loading. Although measurements

of this particular feature were not made before placing the column in the testing machine, continued observations subsequent to the first indicate I think conclusively that this particular deformation existed in the column before loading, and hence that it had no effect upon the ultimate failure of the column, or in other words, that it was an unavoidable result of the processes employed in the manufacture of the column and was not a true buckling of the plates under loading.

The column was accurately placed in the machine with four fine wires stretched throughout its length in the general plane of the upper flanges and with two similarly placed in relation to the lower flanges. These fine wires stretched with constant weights enabled any vertical or horizontal deflections of the tops of the four ribs and the bottoms of the two exterior ribs to be measured by the aid of finely graduated steel scales. Furthermore, longitudinal timber scantlings on the two centre lines of the exterior ribs, carrying steel scales at their ends, were used to measure the shortening of the column under loading for 16 feet of its length to 1/128 of an inch. While these methods of measurements were not so refined as it might be desirable to adopt in an extended series of tests of this nature, they answered well the purposes of this particular investigation, which was not intended so much to determine with refined accuracy all the deformations produced in the test as to discover the main features and methods or other circumstances of failure, so far as possible, which attended the collapse of the full size chord section.

Prick punch marks were made in the heads of the rivets of the lattice bars throughout the length of the upper side of the column as it lay in the testing machine, and the distances between these marked were accurately measured at all stages of the test up to failure in order to ascertain the condition of stress in the lattice angles under the progressive loading to which the column was subjected. Furthermore, these bars were tapped with a hammer at the same time in order to secure further information as to their condition of stress as the tone of the resulting sounds might give.

The progressive loading was applied in stages of 3,000 lbs. per square inch of cross section of column, beginning with an initial loading of that value. At the end of every other stage of each loading, the column was relieved of stress in order to make observations in that condition. This programme was adhered to up to a stress of 21,000 lbs. per square inch, when the next increment was made 1,500 lbs. per square inch, after which the column was freed of load. The remaining programme of loading is shown on the blue print plan showing the effect of the test on the column which will be discussed in full below.

After the application of each 3,000 lbs., or finally 1,500 lbs., increment of loading and upon each removal of loading an accurate series of measurements for shortening of the column, for horizontal and vertical deflections at the various panel points of the latticing and for the stretching or shortening of the latticing angles were made. The results of these measurements are shown on the blue print plan showing the effect of test and largely in the tabular statement on that blue print headed 'Changes in Chord Lengths According to Loading.' The only exception to this statement is the fact that the measured deflections of the columns are not given. As these deflections were small the methods of measuring them were not altogether conclusive as to their amounts or as to their actual existence in some cases. At 9,000 lbs. per square inch, for instance, three ribs showed an apparent upward deflection of $\frac{1}{32}$ inch at and in the vicinity of the centre of the column. This deflection did not appear to increase until the stress reached 18,000 lbs. per square inch, and then only to an amount less than $\frac{1}{16}$ inch with doubt as to the accuracy of the measurement. No apparent increase of deflection was found again until a stress of 24,000 lbs. per square inch was reached, when the deflection of the four ribs appears to be $\frac{3}{32}$ inch, $\frac{1}{32}$ inch, $\frac{3}{32}$ inch and $\frac{1}{8}$ inch, respectively, at centre. On removal of the load this deflection disappeared entirely except for $\frac{1}{32}$ inch in one interior rib and the same amount in one exterior rib, both being upward. There was no subsequent opportunity to make further deflection measurements.

Under a stress of 12,000 lbs. per square inch one rivet in a lattice angle at the second centre intersection from the west end of the column was found loose, but toward the end of the test it appeared to become less so, the conditions of the centre ribs probably becoming such as to give it less opportunity for small motion. Up to the final loading all other rivets appeared to remain in good condition although they were frequently tested with a light hammer.

Actual testing of the column with the application of the first loading began at about two o'clock in the afternoon of November 21, of the current year, and it was continued without interruption in the manner set forth in the preceding statements to 11 p.m. of the same day. At that time a load of 25,000 lbs. per square inch was reached for a very short time in the endeavour to attain a stress of 25,500 lbs. per square inch. This endeavour, however, was unsuccessful in consequence of the leaking of a pump valve (subsequently repaired) to such an extent as to render it impossible to secure the desired pressure in the cylinder of the testing machine.

After having attained the above loading of 25,000 lbs. per square inch the programme of the test was interrupted until 10 a.m. of November 22.

At that time instructions were given to load the column to 25,500 lbs. per square inch, but inadvertence in signalling to attendants at the pump caused the load to reach 26,850 lbs. per square inch, at which stress the member suddenly failed. This failure was attended by a quick sharp report, and it occurred so suddenly that three observers who were closely watching the column at the time could not discover any sequence in the yielding of the details of the column; the occurrence was so sudden that all failures of details appeared to be absolutely simultaneous.

Aside from the raising of scale on the pin plates immediately in front of the 12-inch pins, the collapse of the column consisted in the failure by shearing of the majority of the lattice rivets at the central panel of latticing and of a considerable number of other rivets throughout the length of the column in both flanges, loading to the permanent bending to reversed curvature of the four ribs at the same central vicinity accompanied by the violent bending or distortion of the lattice angles and some small dishing of the rib web plates, all as shown on the accompanying blue print. The ribs were all slightly bent immediately beyond the supporting influence of the battens at each end.

There are certain features of this practically instantaneous failure of the column which are highly significant. As indicated in the preceding statements, there were no permanent strains or distortions of any kind discovered or apparently discoverable up to the loading producing failure. This observation is certainly true of every part of the column except the $\frac{7}{24}$-inch lattice rivets. If suitable apparatus for refined measurements could have been applied to them some shear distortion might have been observed prior to the final loading. Observations made on the latticed angles showed no permanent stretching or compression of those members prior to failure. The phenomenally high elastic limit of the metal in them shown by the test results in the tabular statement and already remarked upon indicates that they would have exhibited no marked permanent distortion much short of ultimate resistance either as tension or compression members. In point of fact all the circumstances of the test indicated that no main part of the column was stressed up to its elastic limit; in other words, that the entire loading was insufficient to develop more than a part of the elastic resistance of the column as a whole, and that if the latticing details had been stronger the column would have carried a greater load before collapsing. The instantaneous failure was clearly due to the fact that the main parts of the column were subject to elastic stress only.

Although it is impossible to corelate accurately the results of this column test with the stress conditions in the actual chord section at the instant of failure, in consequence of the higher elastic qualities of the relatively thin metal in the model column which has already been commented upon and the presumably greater care

which is usually bestowed upon the manufacture of a model member, an approximation of some value may perhaps be made.

The friction of a hydraulic testing machine is known to be considerable, but without recent calibration its amount cannot be confidently stated. The friction of the machine at the Phœnix Bridge Company's works was determined some ten or twelve years ago by G. Henning as $17\frac{1}{2}$ per cent of the total load on the piston as indicated by the mercury gauge, and this may be accepted provisionally until a further calibration can be made. If this percentage deduction be made from 26,800 lbs. per square inch, the apparent stress at which the column failed, it will make the compressive stress in the metal 22,110 lbs. per square inch. The shear tests of the $\frac{7}{24}$-inch and $\frac{7}{8}$-inch rivets make the average of the latter but 86 per cent of the former. Hence, if the ultimate shearing resistance of the $\frac{7}{24}$-inch rivets had been the same as that of the $\frac{7}{8}$-inch rivets, the stress on the column producing the failure of the latticing rivets would have been but 19,014 lbs. per square inch of the column. Just what value should be given to the possibly higher excellence of manufacture of the model over that of the full size column is of course not determinable. It may or may not have sensible value. It is to be noted, however, that after making such allowance as is practicable for the friction and the increased resistance of the smaller rivets there is reached an intensity of stress nearly identical with that which existed in the actual chord section at the time of this failure.

It should be carefully observed that the radius of gyration of the normal section of the model column about an axis at right angles to the webs and through their centres, i.e. parallel to the axis of each pin, is 5·43 and 5·52 inch about a central axis parallel to the webs. Hence, as the column lay in the testing machine, the ratio of its length divided by the horizontal radius of gyration is 35, while the ratio of the same length over the vertical radius of gyration is 42. The column failed, therefore, in the plane of the greatest radius of gyration. Furthermore, its failure was wholly in a horizontal plane, there being no sensible vertical deflection of the failed column.

The length of the column was such as to place it practically at the limit between short and long columns, as the ordinary column formulæ, such as the much used Gordon's and 'straight line,' are properly applicable when the ratio of length over radius of gyration has values greater than about 40 or possibly a little more. Inasmuch as the ultimate carrying capacity per square inch of section increases as the length of column decreases and as this model column was comparatively short, the latticing required to develop its full load carrying power should be relatively heavy rather than light.

Very truly yours,

WM. H. BURR,
Cons. Engineer.

The commissioners were invited to be present and to assist at this test, and the Department of Railways and Canals was represented at it by Mr. C. C. Schneider. The shape of the model chord after the test was finished is shown on drawing No. 21, which has been prepared from the blue prints referred to by Prof. Burr. The accompanying photographs (Nos. 1 and 2) show the details of the failure very clearly.

The commission has the following comments to make concerning this test:—

1. There was little or no indication of failure up to the instant at which it occurred. Failure took place with explosive violence, by the shearing of the outer rivets of the latticing.

2. Messrs. Schneider, Deans and Szlapka were closely watching the chord when the unexpected failure occurred. No one of these engineers could say what connection or detail was the first to give way.

3. It was noted that the surface scaled at only three outside pin bearings.

4. The cross lattice bars showed little sign of stress, rivets being sheared only in those bounding the central bay. Theoretically, under a racking stress in the column, these bars would not come into play, the diagonals only being strained, one set in tension and the other set in compression.

5. The failure of the riveting was systematic. In each lattice bay one diagonal connection failed in tension and the other in compression; and usually both failures occurred at the side of the bay farthest from the nearer end of the column. It will be noted from the photographs that both on top and bottom the diagonals in the centre bay failed in the opposite manner to that of the corresponding diagonals in all the other bays.

6. The efficiency of the central connection plate in the bottom latticing is well shown by the photographs.

7. Some of the lattice rivets were cut out and found to be partially sheared and there were some slight indications that they had been sheared first on one side and then on the other, indicating a reversal of stress in the lattices. That such a reversal would instantly follow the failure of the latticing in the central panel is apparent from the curvature of the chord on one side of the central panel being opposite to that on the other side.

8. Subsequent investigation has shown that the method adopted for testing the working of the lattice bars was unsatisfactory. It consisted of measurements between centre punch marks on the rivet heads and did not include the effect of rivet shear.

In our opinion the load was more evenly and centrally applied in this test than it would be in the case of a chord in ordinary service. In other words under working conditions the failure of the latticing of this model chord would have taken place under a smaller stress.

This is the more probable since, as Prof. Burr points out, the model chord was superior to the bridge chord in both material and workmanship. The difference in material is well shown by the test records included in Prof. Burr's report.

On November 26 some tests on rivet shear were made by the Phoenix Bridge Company, the results being given on Drawing No. 26. The results of these tests, together with those given by Prof. Burr, showed that the rivets used in the bridge would develop an ultimate strength of slightly over 50,000 lbs. per sq. inch, and also that the rivets maintained without failure their ultimate strength, even though partially sheared.

On January 14 some further tests were made by the Phoenix Bridge Company which gave similar results. These results are given on Drawing No. 26. A movement of $\frac{3}{16}$ to $\frac{1}{4}$-inch apparently took place before actual failure.

This rivet shear offered a reasonable explanation of part of the change in the lengths of the diagonal lattice bars which must have accompanied the distortions of the chords in the bridge which were measured on August 27, 1907. The inspectors had indeed carefully examined the chords and the lattices and reported no evidence of failure, but this rivet shear might easily have escaped their observation; it is noteworthy that no one of the engineers assembled to watch the test of the model chord on November 21 thought of it, and they completely failed to detect such action up to the moment of failure, although working under the most favourable conditions. A change in length in addition to the above seemed to be due to the reduced section at the centre of one of each pair of diagonal lattice bars.

In December the commission ordered the construction of test chord No. 2 for the purpose of determining the strength of the webs of the design used in the Quebec bridge. The dimensions of this chord are given in Drawing No. 23. It had a section half that of test chord No. 1; the number of rivets in the lattice connections was doubled, the section of the lattice bars was increased 50% and the weak points at their centres were strengthened by the use of connection plates. The webs were of the same section as the outer webs of test chord No. 1. Material from the same heats was used in the manufacture of the two test chords.

This chord was tested at Phœnixville on January 18, the observations made during the test and the shape of the chord after failure being shown on Drawing No. 24 and on photographs Nos. 3 and 4.

It will be noted that this chord failed under a stress of 37,000 lbs. per sq. in. by the buckling of the webs in the centre bay, the latticing being sufficiently strong to fully develop the strength of the webs. The nominal strength of the column (the record has to be corrected for an unknown machine error) was slightly less than the elastic limit of the metal in the webs. (See record in Professor Burr's report.)

In this test the column seems to have been loaded evenly and centrally, since the latticing was not seriously stressed :

The following notes concerning this test are of interest :—

(1) There was some reason to think that chord A—9—L might have been bent sharply at the edge of the cover plate previous to failure. The inclination of its webs to the centre line on August 27 was very marked near the 8-9 joint. (*See* Drawing No. 28). A series of straight edge measurements (*see* Drawing No. 24) was made during the test of model chord No. 2 to determine whether any angle developed at the edge of the cover plate as the pressure increased, but no such movement was detected.

(2) During the test the yielding of the lattice rivets was observed by means of match marks upon the lattices and ribs. The results noted are given on drawing No. 24. They indicate that the pressure was centrally applied and that the lattice bars were not seriously stressed when these observations were taken. Towards the end of the test the lattice bars in the end panels were distinctly bowed upwards owing probably to the compression of the webs.

(3) Scaling at the pin bearings was observed as shown on Drawing No. 24, but this was about the same at all four surfaces at the bearings and did not indicate that there was any racking strain on the chord.

(4) The dishing of the webs during the test is shown on Drawing No. 24.

(5) It will be noted from the measurements on Drawing No. 24 that there were practically no horizontal or vertical movements of the chord webs with reference to the chord ends to which the reference wires were attached.

On January 20, the Commission made three tests on full size lattice bars of the chord No. 9 design, the particulars of which are given on Drawing No. 27. The tests were made in the laboratory of Messrs. Wm. Sellers & Company, Philadelphia, and the results obtained under the skilful handling of Mr. Backstrom may be accepted without question.

The purpose of these tests was to determine the strength of the lattice bars and the amount of yielding of the various parts of the length as the tension increased.

It will be noted that in each case failure took place at the centre of the lattice bar and the following table is of interest :—

RESULTS OF LATTICE AND RIVET TESTS.

3 full size bars tested by Wm. Sellers & Co. on Jan. 20th, 1908.	3 short sections of full sized bars tested by the Phoenix Bridge Co. on Jan. 21st, 1908.	4 tests on $2\frac{7}{8}$-in. rivets in single shear made by the Phœnix Bridge Co. on Nov. 26th 1907.	2 tests on $2\frac{7}{8}$-in. rivets in single shear made by the Phœnix Bridge Co. on Jany. 14th, 1908.	3 tests upon $2\frac{7}{8}$-in. rivets in double shear, made by the Phœnix Bridge Co. Nov. 1907.
Ultimate load in lbs.	Ultimate load in lbs.	Ultimate load in lbs.	Ultimate load in lbs.	Ultimate load in lbs.
60,100	61,100	63,000	62,500	121,000
59,800	62,000	63,000	63,100	122,000
59,500	60,700	63,800	....	123,800
		64,700		

It will be noted from the above that the riveting was sufficiently strong to develop the full strength of the cut lattice bar and by reference to Drawing No. 27 it will be seen that yielding took place simultaneously in the rivet connections and in the reduced section at the centre of the bar.

As the first test chord failed by shearing of the rivet connections and as no indications of failure at the centre of the lattice bars were noticed, the result of the tests on the full size lattice bars was unexpected.

A series of tests on the lattice bars of test chord No. 1 was made by direction of the commission on January 23rd. The small testing machine belonging to the Phœnix Iron Company was used, but as this was not well equipped for the work, the results are not wholly satisfactory.

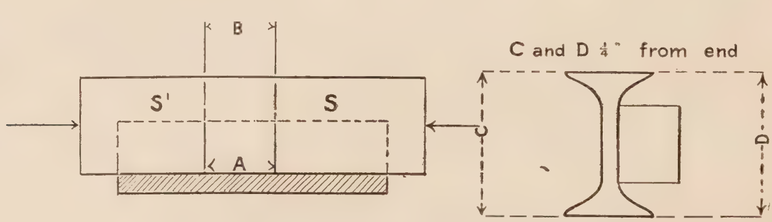
The observations which are given on Drawing No. 25, show that up to the moment of failure there was little yielding either of the rivets or of the reduced central portion, and that the failure took place in each case by rivet shear. The results of the specimen tests on the material for the angles are given in Prof. Burr's report.

The following table gives the results of tests upon the $\frac{7}{24}$-in. rivets.

TESTS ON $\frac{7}{24}$-IN. RIVETS.

At Phœnixville, November, 1907. 2 Rivets in Double Shear.	At Phœnixville, Jan. 21st 1908. Two Rivets in Single Shear.
Ultimate load in lbs.	Ultimate load in lbs.
16,000	7,500
15,600	8,700
16,000	9,000

On January 31, the Commission made some tests at the laboratory of Messrs. Wm. Sellers and Company to determine the slips of rivets connecting parts under compression, the form of the test pieces before and after testing being shown on drawing No. 25. The record of these tests, which is not given elsewhere, is as follows:—



TESTS No. 1874.—JANUARY 21, 1908.

Loads.	Distance A.	Distance B.	Distance C.	Distance D	Remarks.
	Ins.	Ins.	Ins.	Ins.	
0	1·0069	1·2700	5·90	6·04	
5,000	1·0050	1·2623	5·90	6·04	
10,000	1·0018	1·2612	5·90	6·04	
15,000	1·0000	1·2529	5·90	6·04	
20,000	1·0000	1·2505	5·88	6·05	
25,000	·9975	1·2469	5·88	6·05	
30,000	·9940	1·2304	5·86+	6·06	
35,000	·9820	1·2178	5·86+	6·07	
40,000	·9391	1·1570	5·86+	6·07	
40,000	·9390	1·1568	5·86+	6·07	After 10 minutes rest.
45,000	·8912	1·0982	5·87	6·08	
50,000	·8228	1·0090	5·84	6·09	
50.000	·8050	·9913	5·84	6·09	After 10 minutes rest.
55,000	·7090	·8470	5·80	6·11	I beam starting to scale.
59,200		Maximum load reached.			Structure collapsing.
0	·5271	·4106	5·46	6.31	

Motion of Blocks S and S′
S =″200+
S′=″180

TEST No. 1875.

Loads.	Distance A.	Distance B.	Distance C.	Distance D.	Remarks.
	Ins.	Ins.	Ins.	Ins.	
0	1 0018	1·1941	5·98	5·96	
5,000	1·0010	1·1941	5·98	5·96	
10,000	1·0010	1·1896	5 98	5·96	
15,000	·9991	1·1830	5·98	5·96	
20,000	·9991	1·1797	5·98	5·96	
25,000	·9973	1·1738	5·98	5·96	
30,000	·9905	1·1633	5·98	5·96	
35,000	·9852	1·1528	6·98	5·96	
40,000	·9411	1·1004	5·96	5·96	
40,000	·9411	1·1004	5·96	5·96	After 10 minutes rest.
45,000	·9072	1·0536	5·95	5·98	
50,000	·8367	·9569	5·93	5·99	
50,000	·8318	·9535	5·93	5·99	After 10 minutes rest.
55,000	·6953	·7647	5·90	6·03	I beam scaling.
58,700		Maximum load reached.			Structure collapsing.
0	·5195	·3760	5·52	6·25	

Motion of Blocks S and S′
S =210″
S′=230″

Measurements C and D are on rough surfaces.

GUS E. BACKSTROM.

TEST No. 1873.

Load.	Distance A.	Distance B.	Remarks.
0	1·0206	... . .	
5,000	1·0206	....	
10,000	1·0206	....	
15,000	1·1074	....	
20,000	1·0150	1·2762	
25,000	1·0143	....	
30,000	1 0097	....	
35,000	1·0002	. . ..	
40,000	0·9730	....	
45,000	0·9184	1·1176	
45,000	0·9130	1·1455	(12 mins.)
45,000	0·9134	1·1455	After 10 minutes sustained load.
50,000	0·8390	1·0529	After 15 minutes sustained load.
50,000	0·8303	1·0430	After 10 minutes sustained load.
55,000	0·6963	0·8520	Beam begins to scale.
57,600	... .	. ...	Maximum load reached then falling off from distortion of I beam.

Block slip 0·18″ and 0·211″ after completion of test.

It will be noted that under light loads the slip was much the same as in tension tests and that as the loading increased the web of the I beam and not the riveting gave way.

These tests were made for the purpose of obtaining information to throw light upon the fracture of chord A 9-L Quebec Bridge. The discussion of the failure of this chord will be found in appendix No. 16.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY,
J. GALBRAITH.

APPENDIX No. 16.

A DISCUSSION OF THE THEORY OF BUILT-UP COMPRESSION MEMBERS.

This discussion will be confined to columns of which the cross section is rectangular in outline and which are built up of two or more parallel webs with stiffening angles, connected by lattice bars, tie plates, diaphragms, etc. In such columns the parallel webs carry the load and the connections serve a subsidiary purpose. For convenience the webs, considered apart from their connections, will be termed the web system and the connections the lattice system. In many bridges the continuous cover plates of the top chord belong to both the web system and the lattice system, inasmuch as they both carry load and serve as connections for the side plates.

In the design of the cross section the arrangement and dimensions of the web system are first considered. Column formulas based on experiment are used for this purpose. These formulas give the average unit stresses under which columns fail in terms of length and radius of gyration. This radius is taken in the plane in which the column will probably fail by buckling or bending. A factor of safety is used in the design and a suitable arrangement of the cross section of the web system adopted. The web system, in short, is designed from column formulas or the plotted results of experiments from which these formulas are deduced.

The design of the lattice system is quite a different matter. As a rule it depends on the judgment of the engineer guided solely by experience. He finds little or nothing in scientific text books or periodicals to assist his judgment. Some lattice formulas are in existence, but they are not generally known and their utility is more or less doubtful owing to the uncertainty of the data and assumptions on which they are founded. The unsatisfactory nature of the column formulas upon which the web system is designed is a matter of common knowledge among engineers, but the column formulas may be considered to represent exact science in comparison with the lattice formulas.

The lattice system performs two distinct functions. In the case in which each web of the web system carries its share of the load, that is to say when there is no transfer of load in any part of the column from one web to another, the lattice system simply acts as a side support to the web system and by means of it a long web is divided up into a number of short columns. The stresses thrown into the latticing in this state cannot be computed. In this case the load on the column is parallel to the axis but not necessarily coincident with it, and the curvature is assumed to be negligible. When, however, the load is inclined to the axis of the column, the lattice system has a different function. The angle of inclination may vary from point to point along the column owing to the curvature of the column. This curvature may be due to original bends or to the action of the load or to both combined. If the curvature is sufficiently small the variation of inclination due to it will be negligible. There remains, however, the original inclination or obliquity which is due to the method of application of the loads at the ends of the column. If the eccentricity of application is the same at each end and in the same plane with the axis of the column, there will be no obliquity other than that arising from the curvature of the webs which may be negligible. If, however, the eccentricities at the opposite ends are different or in different directions the obliquity may be of considerable amount. If the curvature of the column be negligible the obliquity arising from the eccentricity will be the same at every point. This obliquity causes a transfer of load throughout the whole length of the column from one web to another. This transfer of load is accompanied by longitudinal shearing stresses in the lattice system. The obliquity also causes transverse shearing stresses at every cross section of the column.

If the lattice system is considered to be sufficiently stiff the longitudinal shearing forces can be derived from the transverse shearing forces by the ordinary processes of statics as applied to elastic solids, and from them the lattice stresses and the lattice cross sections may be computed.

If θ is the angle between the direction of the column axis and that of the load, S the transverse shear and P the load,

$$S = P \sin \theta$$

and since in practical cases θ is small this may be written

$$S = P \ \theta$$

if θ be expressed in radians or as the ratio of the total eccentricity to the length of the column.

This formula holds true also if the curvature of the column is great enough to require consideration. In such a case θ varies along the column, and in computations the column should be divided by cross-sections so close together that the difference in θ at two neighbouring cross-sections may be disregarded.

So far the problem is comparatively easy—with the next step difficulties begin. The question now is,—what value of the obliquity shall be chosen in design?

Since the obliquity depends upon inequality in the eccentricities at the ends, the maximum difference must be decided upon for design. It would seem reasonable to assume for this purpose equal eccentricities in opposite directions so that if e be

the assumed eccentricity at one end the maximum value of the obliquity will be given by

$$\theta = \frac{2\,e}{l}$$

l being the length of the column. Against this view, however, it may be urged that the chances of the maximum value ever being reached are extremely small and that therefore some smaller value should be chosen.

Evidently the strength of this objection depends upon the value assumed for the eccentricity. The safe maximum value to be assumed for e depends upon the excellence of the design both of the column and of the splices, on the accuracy of workmanship, and on the care and precision employed in the erection.

It is impossible to estimate with accuracy the value of e under any set of conditions, but reasonable limits for its values will doubtless be learned from experience and study. With bad work, and more especially bad fitting and weak splices at butt joints, the value of e may be much greater than it need be under other conditions of construction. In design, however, good workmanship and strong splices should be assumed. Theoretically the cross sections of the latticing should be designed so that with the assumed eccentricity the lattice and the web systems will get their ultimate safe stresses simultaneously. This condition will be satisfied if the unit stress in the latticing has the same factor of safety as the maximum compressive stress in the web system corresponding to the eccentricity.

Let P be the safe load, A the area of the cross-section $p = \frac{P}{A}$, d the greatest diameter of the cross-section in the plane of the latticing, r its radius of gyration parallel to d, q the unit stress at the most compressed edge, e the eccentricity of the load P, then

$$q = p\left(1 + \frac{e\,d}{2\,r^2}\right)$$

an equation which is generally true only within the limit of elasticity—and consequently

$$e = \frac{2\,r^2}{d}\;\frac{q-p}{p}$$

$$\theta = \frac{2\,e}{l} = \frac{2}{l}\;\frac{2\,r^2}{d}\;\frac{q-p}{p}$$

$$S = P\,\theta = p\,A\,\frac{2}{l}\;\frac{2\,r^2}{d}\cdot\frac{q-p}{p}$$

$$= A\,\frac{2}{l}\;\frac{2\,r^2}{d}\,(q-p)$$

In design all the quantities in the above expressions for e, θ, and S are fixed without difficulty with the exception of q, the extreme unit stress in the web system. It is evident from the formula that S becomes zero when $q = p$. Now the maximum value of q for which the formula

$$q = p\left(1 + \frac{e\,d}{2\,r^2}\right)$$

holds, is in general, the elastic limit. Consequently as p approaches the elastic limit, S approaches zero. Evidently, when p is equal to the elastic limit, the load must be central and without obliquity since no part of it can be transferred from one web to another without inducing stresses in the second web in excess of the elastic limit.

The function of the latticing in such a case is simply to stiffen the webs and, as has already been said, the accompanying lattice stresses cannot be computed. The condition necessary for a theoretical computation of the stresses in the latticing is

that the difference between p and q be reasonably large. Lattice formulas of course fix a value of q in a fashion, but only tests and experience can determine whether or not these give economical and safe results. Direct tests are difficult to apply and unless great care is exercised, incorrect inferences may be drawn from them. A lattice column placed in a testing machine may fail in the web system, but this is no indication that the lattice system is strong enough for service in a similar column when in use as a bridge member. It may be that the obliquity of the load was too small to develop the lattice strength. With a greater obliquity the column might fail in the lattice system under a much smaller load. In other words, the failure of the webs is an indication that the full strength of the column has been nearly developed. The failure of the latticing may not be such an indication. The full strength of the column can be developed only by axial loading, and under such loading comparatively weak latticing may serve to develop this strength.

The full strength of the latticing can be developed only by oblique loading. The column strength in this case must be less than under axial loading.

The case of lower chord $A\ 9\ L$ Quebec bridge is an example of an insufficient lattice system. The webs bent and the lattices failed under a load only three-fourths of the specified maximum working load.

DETERMINATION OF THE AREA OF A LATTICE BAR CROSS-SECTION.

The bar must be designed to take equal stresses in tension and compression. Let P' represent the lattice stress, A' the tension section, A'' the compression section q' the unit stress in tension, q'' that in compression. The unit stress q'' must be computed by a column formula.

Now $P' = k\ S$, where k is a coefficient which can be calculated from the known arrangement and dimensions of the lattice and web system. This calculation will be taken up later.

Thus

$$P' = A'\ q' = A''\ q'' = k\ S$$

$$\therefore\ A' = \frac{k\ S}{q'} = k\ A\ \frac{2}{l}\ \frac{2\ r^2}{d}\ \frac{q-p}{q'}$$

$$A'' = \frac{k\ S}{q''} = k\ A\ \frac{2}{l}\ \frac{2\ r^2}{d}\ \frac{q-p}{q''}$$

q' and q'' in design should have at least the same factor of safety as q.

It may be more convenient in many cases to make A' represent the shearing area of the rivets and q' the shearing unit stress. The net area of the lattice bar has been selected in this discussion because in the Quebec bridge it was weaker than the rivet areas, the lattice bar section being 1·15 sq. ins. and the rivet area 1·80 square inches (3 rivets).

In the arrangement of the lattice system the free portions of the webs should have a value of $\frac{l}{r}$ less than that of the column as a whole.

LATTICE FORMULAS.

In the foregoing discussion it has been shown that there are two points with regard to which there must be more or less doubt and in which no aid can be expected from theory; first, the stresses to which the latticing is exposed when the load is axial and, second, the value to be assumed for the maximum unit stress in the web system.

The assumption is made that a satisfactory solution of the second difficulty is sufficient to provide for the first. All practical lattice formulas determine, in effect, the value to be assigned to q. The same factor of safety is used for q' and q'' as for q.

Mr. C. C. Schneider, Consulting Engineer, has called the attention of the Commission to an article on this subject by Professor Prandtl of Goettingen, in the 'Zeitschrift des Vereines deutscher Ingenieure' of December 28, 1907. Professor Prandtl assumes that the equation

$$q = p\left(1 + \frac{e\ d}{2\ r^2}\right)$$

holds up to the point of failure of the outer web. He makes q the ultimate strength of the portion of the outer web between neighbouring lattice points. He necessarily makes q' and q'' in the formula for lattice bars represent ultimate strengths. He also discusses the allowance to be made in the value of r on account of want of stiffness in the latticing. In other respects his discussion corresponds to ours.

In the same journal a letter appears from Professor Engesser of Karlsruhe, who also refers to the diminution of r owing to want of stiffness of the latticing. This letter does not contain sufficient information to enable a reader to follow the line of thought. However, in the formula for lattice bar areas, he seems to replace $\frac{q-p}{q'}$ and $\frac{q-p}{q''}$ by $\frac{q-p}{q}$ in which q is the ultimate strength of the material and p is the ultimate strength of the column which he determines by use of the Tetmajer formula. He also replaces the factor 2 by π.

He states that he published his investigations in 1891 and 1893.

Mr. H. S. Prichard in 'Engineering Record,' October 12, 1907, gives a rule which he had used for several years which makes

$$S = \cdot 015\ P$$

in other words

$$\theta = \cdot 015$$

Mr. Szlapka, in his evidence, states that after a most painstaking search the only information he could find on the subject of lattice computations was that given in Johnson's 'Modern Framed Structures.'

The experience of the Commission is practically similar to that of Mr. Szlapka for, except the rule in 'Modern Framed Structures,' all the information we have been able to find has appeared in the periodical press since the collapse of the Quebec bridge.

Mr. Bindon B. Stoney, one of the earlier authorities on bridge construction, has given a method of computing statically the stresses in lattices based on the assumption of curvature in the column.

The article in 'Modern Framed Structures' is as follows:—

'There are no rules other than empirical ones in use by which the size and spacing of lattice bars for compression members are determined. . . . It has been suggested that, as our compression formulas all assume a certain extreme fibre stress due to the flexure of the strut, from this known extreme fibre stress we find an equivalent uniform load acting in the plane of the latticing which will produce this fibre stress and from this load find the stress in the lattice bars.'

This method is equivalent to assuming that $q = f$ and $\theta = \frac{4\ e}{l}$, f being taken from the formula $p = f - c\frac{l}{r}$ or the formula $p = \dfrac{f}{1 + \frac{1}{c^1}\left(\frac{l}{r}\right)^2}$ when applied to the working stresses.

The value of S thus becomes

$$S = P\ \theta = P\frac{4\ e}{l} = P\frac{8\ r^2}{l\ d}\ \frac{f-p}{p}$$

From the straight line formula $\frac{f-p}{p} = \frac{p}{l}\;\frac{r}{\text{ɔ}} = \frac{A}{P}\;\frac{c\;l}{r}$

So that

$$S = \frac{P.\;8\;r^2\;\;A\;c\;l}{l\;\;\;\;d\;\;P\;r} = \frac{8\;A\;c\;r}{d}$$

From the Rankine formula

$$\frac{f-p}{p} = \frac{1}{c'}\left(\frac{l}{r}\right)^2$$

$$\therefore\; e = \frac{2\;r^2}{d}\;\frac{1}{c'}\left(\frac{l}{r}\right)^2 = \frac{2\;l^2}{c'\;d}$$

$$\theta = \frac{4\;e}{l} = \frac{8\;l}{c'\;d} \qquad\qquad S = P\;\theta = \frac{8\;l}{c'\;d}\,P$$

These equations give very different values for S, even though the constants of the Rankine formula be computed so that the curve represented by it and the straight line represented by the straight line formula are tangent at the point corresponding to the value of $\frac{l}{r}$ in question.

Mr. Szlapka used the rule in 'Modern Framed Structures.' He selected Rankine's formula and gave c' its value for square bearings. He, however, modified the method by using a central load instead of a distributed load. This modification had the effect of making the area of the lattice bar one-half of that given by the method suggested in 'Modern Framed Structures.'

Mr. Szlapka finally adopted a larger cross-section than his method gave, and one which, in his judgment, was sufficient.

If he had tested the method fully he would have found it capable of giving areas ranging up to ten times the area computed by him, a result which would have shown the indefiniteness of this method. He might, of course, have come to the conclusion that a rule capable of giving such different results was valueless.

In an article in 'Engineering,' September 27, 1907, Professor Keelhoff of Ghent University, states that in 1893 he developed a lattice formula which has been more or less extensively used. When thrown into the form of the theoretical formula previously given, the following results are obtained.

Professor Keelhoff multiplies the expression for θ by the coefficient $\frac{\pi}{2}$. Thus, instead of $\theta = \frac{2\;e}{l}$, his method gives $\theta = \frac{\pi\;e}{2}$, this change being the result of a theoretical study in which he adopted Euler's sinusoid curve as the probable form of a bent column. He also replaces q by f taken from the column formula for working stresses

$$p = f - c\,\frac{l}{r}\,.$$

It is thus apparent that the methods of lattice computation which have been used in practice, when thrown into the form adopted in this discussion, simply assign values to the unknown q, and in some cases multiply the theoretical obliquity by the factor 2 or $\frac{\pi}{2}$.

The following is a resume of the lattice formulas discussed in this appendix:—

Theoretical Formulas.

$$e = \frac{2\ r^2}{d}\ \frac{q-p}{p}$$

$$\theta = \frac{2\ e}{l} = \frac{2}{l}\ \frac{2\ r^2}{d}\ \frac{q-p}{p}$$

$$S = P\ \theta = p\ A\ \frac{2}{l}\ \frac{2\ r^2}{d}\ \frac{q-p}{p}$$

$$= A\ \frac{2}{l}\ \frac{2\ r^2}{d}\ (q-p)$$

$$A' = \frac{k\ S}{q'} = k\ A\ \frac{2}{l}\ \frac{2\ r^2}{d}\ \frac{q-p}{q'}$$

$$A'' = \frac{k\ S}{q''} = k\ A\ \frac{2}{l}\ \frac{2\ r^2}{d}\ \frac{q-p}{q''}$$

q to be determined by judgment and not to exceed the elastic limit.
q' and q'' to have the same factor of safety as q.

Formulas used in Practice.

Prandtl makes q the ultimate strength of the portion of the outer web between neighbouring lattice points and q' and q'' the ultimate strength of the lattice bars used.

Engesser replaces $\frac{q-p}{q'}$ and $\frac{q-p}{q''}$ by $\frac{q-p}{q}$ respectively, using ultimate values. He also multiplies θ by $\frac{\pi}{2}$ *i.e.* makes $\theta = \frac{\pi\ e}{l}$.

Prichard makes θ constant $= \cdot 015$.

'Modern Framed Structures' makes $q = f$ of the working stress formula

$$p = f - c\ \frac{l}{r} \text{ or } p = \frac{f}{1 + \frac{1}{c'}\left(\frac{l}{r}\right)^2}$$

And also multiplies θ by 2, *i.e.* makes $\theta = \frac{4\ e}{l}$

Szlapka modified the rule in 'Modern Framed Structures' by not using the multiplier 2 in the value of θ, *i.e.* made $\theta = \frac{2\ e}{l}$

and also used the formula

$$p = \frac{f}{1 + \frac{1}{c'}\left(\frac{l}{r}\right)^2}$$

giving c' its largest value, viz.:—36,000.

Keelhoff makes $q = f$ of the working stress formula $p = f - c\ \frac{l}{r}$

and also multiplies θ by $\frac{\pi}{2}$

Computation of k in formula $P' = k\ S$.

The method of making this computation will be illustrated by a numerical example.

For this purpose lower chord A 9—L Quebec bridge will be taken.

On the assumption that the latticing is sufficiently stiff to enable the webs to act as a unit, the relation between the longitudinal shear S' in the length of one panel of latticing and the transverse shear S at the end of the panel is given by the statical formula $S' = \frac{S\,x\,Q}{I}$ where x is the length of one lattice panel, Q the moment of area about the central axis of the chord cross-section perpendicular to the lattice planes of that portion of the web cross-section which lies outside the given plane of longitudinal shear, and I the moment of inertia of the whole chord cross-section about the same axis.

Evidently maximum S' corresponds to maximum Q which, in chord A 9-L, occurs between the centre webs. The numerical values are $Q = 6439$. $x = 72{\cdot}75$. $I = 302640$, dimensions being given in inches.

Therefore $S' = 1{\cdot}55\ S$ between the centre webs. Similarly between the outside web and the centre web $Q = 5313$, giving

$S' = 1{\cdot}28\ S$

In a lattice panel there are four bars arranged two and two as the diagonals of a square of which the side is 54·36 inches, this being the distance between the axes of the outside webs.

Therefore $P' = \frac{S'}{4} \times \sqrt{2} = {\cdot}35\ S'$

$$P' = {\cdot}35 \times 1{\cdot}55\ S = {\cdot}54\ S \text{ between the centre webs}$$

$$\text{and } P' = {\cdot}35 \times 1{\cdot}28\ S = {\cdot}45\ S \text{ between an outside web and a centre web.}$$

Thus the values of k are ·54 and ·45.

From the design of the chord it is evident that the net area of the lattice bar is governed by $k = {\cdot}54$, while the rivets connecting the bar to the outside web are determined by $k = {\cdot}45$ and those connecting the bar with the inside web by the difference of these values, which is ·09. That is to say, if 5 rivets were necessary to connect the bar to the outer web, only one would be required for the connection with the inner web.

Transverse shears and bending moments exist in the webs due to the transverse shear S on the cross-section of the chord.

The maximum transverse shear in the outer web occurs in the space between two consecutive panels of latticing. It is equal to $\frac{1}{2} \times \frac{54{\cdot}36}{72{\cdot}75} \times 1{\cdot}28\ S = \frac{1}{2} \times {\cdot}747 \times 1{\cdot}28\ S = {\cdot}48\ S$, if the small bending moments in the webs due to the assumed distribution of stress at the plane of section be neglected; the maximum shearing stress on a centre web section is thus ·02 S, the sum being ·50 S, half the shearing stress on the cross-section of the chord.

Thus 96 per cent of the transverse shear is carried by the outer webs and only 4 per cent by the inner webs in the space between the panels.

The difficulty of determining theoretically the values to be assigned to the quantities $\frac{q-p}{q'}, \frac{q-p}{q''}$ in the formulas

$$A' = k\ A\ \frac{2}{l}\ \frac{2\ r^2}{d}\ \frac{q-p}{q'}$$

$$\text{and } A'' = k\ A\ \frac{2}{l}\ \frac{2\ r^2}{d}\ \frac{q-p}{q''}$$

has been pointed out. It will be of interest to compare the solutions of this problem as given by the various methods that have been described by means of a numerical example and also to compare the corresponding lattice cross-sections.

For this purpose the web system of lower chord A 9-L, Quebec bridge, will be selected. It will be suffcient for the present purpose to consider the formula for the tension section A'.

In this chord, $l = 684$ inches, $r = 19{\cdot}7$ inches. $d = 67{\cdot}5$ inches. $A = 780$ sq. in. $k = {\cdot}54$

Thus—

$$A' = {\cdot}54 \times 780 \times \frac{2}{684} \times \frac{2 \times 19{\cdot}7^2}{67{\cdot}5} \times \frac{q-p}{q'}$$

$$= 14 \frac{q-p}{q'} \text{ sq. ins.}$$

Prandtl: $\frac{l}{r}$ for outer web between lattice points $= 44$; ultimate strength of outer web say $48{,}000 - 210 \times 44 = 38{,}760$.

Specified unit load on column $p = 24{,}000$.

Tensile strength of lattice bar $q' = 60{,}000$.

$$\therefore \frac{q-p}{q'} = \frac{38760 - 24000}{60000} = \frac{14760}{60000} = {\cdot}25$$

$$A' = 14 \times {\cdot}25 = 3{\cdot}50 \text{ sq. ins.}$$

If the unit load for the column had been determined by the formula $p = 16000 - 70\frac{l}{r}$ $= 16000 - 70 \times 34{\cdot}7 = 13571$, we should have had

$$\frac{q-p}{q'} = \frac{38760 - 13571}{60000} = \frac{25189}{60000} = {\cdot}42$$

and $A' = 14 \times {\cdot}42 = 5{\cdot}88$ sq. ins.

Engesser:—

$$q = q' = 60000$$

$$p = 48000 - 210 \frac{l}{r} = 48000 - 210 \times 34{\cdot}7 = 40713$$

$$\frac{q-p}{q'} = \frac{60000 - 40713}{60000} = \frac{19287}{60000} = {\cdot}32$$

$$A' = \frac{\pi}{2} \times 14 \times {\cdot}32 = 7{\cdot}04 \text{ sq. ins.}$$

When this formula is used, the cross-section of the latticing does not vary with the load.

Prichard:—

$$\theta = {\cdot}015$$

Now theoretically $\theta = \frac{2}{l} \frac{2\,r^2}{d} \frac{q-p}{p}$

$$= \frac{2}{684} \times \frac{2 \times 19{\cdot}7^2}{67{\cdot}5} \times \frac{q-p}{p}$$

$$= {\cdot}0333 \frac{q-p}{p}$$

$$\therefore \frac{q-p}{p} \quad \frac{{\cdot}0150}{{\cdot}0333} = {\cdot}45$$

$$\therefore \frac{q-p}{q'} = \frac{q-p}{p} \frac{p}{q'} = {\cdot}45 \frac{p}{q'} = \frac{{\cdot}45 \times 24000}{40000} = {\cdot}27$$

Assuming that 40000 in tension represents the same factor of safety as 24000 (in the case of this column) in compression,

$$A' = 14 \times \frac{q-p}{q'} = 14 \times \cdot 27$$
$$= 3\cdot 78 \text{ sp. ins.}$$

since $\frac{p}{q'}$ is constant for all factors of safety this result applies to all loads.

'Modern Framed Structures':—

(1) Straight line formula.

$q = 24000 \qquad p = 24000 \quad 105 \times 34\cdot 7 = 20356$
$q' = 30000$

$$\therefore \frac{q-p}{q'} = \frac{24000-20356}{30000} = \frac{3644}{30000} = \cdot 12$$

$$A' = 2 \times 14 \times \frac{q-p}{q'} = 28 \times \cdot 12 = 3\cdot 36 \text{ sq. ins.}$$

In this formula evidently $\frac{q-p}{q'}$ is constant for all factors of safety and therefore for all loads.

(2) Rankine's formula.

$$\frac{q-p}{q'} = \frac{f-p}{q'} = \frac{\frac{p}{c'}\left(\frac{l}{r}\right)^2}{q'}$$

assuming $p = 24000$, $c' = 18000$, $\frac{l}{r} = 34\cdot 7$, $q' = 30000$

$$\frac{q-p}{q'} = \cdot 0535$$

$$A' = 2 \times 14 \times \frac{q-p}{q'} = 28 \times \cdot 0535 = 1\cdot 50 \text{ sq. ins.}$$

if c' be made 36000 the values are

$$\frac{q-p}{q'} = \cdot 0267 \qquad A' = \cdot 75 \text{ sq. ins.}$$

Evidently the same results will be given by all factors of safety *i.e.,* for all loads.

Mr. Szlapka's method, if he had used the proper value of k, would have given $A' = \cdot 37$ sq. ins. He assumed that the panels of latticing were square, whereas they were oblong and the lattice bars were not diagonals.

In the actual design, however, he made $A' = 1\cdot 15$ sq. ins.

Keelhoff:—

$$\frac{q-p}{q'} = \frac{f-p}{q'} = \frac{24000 - 105 \times 34\cdot 7}{30000} = \frac{24000-20356}{30000} = \frac{3644}{30000} = \cdot 12$$

$$A' = \frac{\pi}{2} \times 14 \times \frac{q-p}{q'}$$
$$= 1\cdot 57 \times 14 \times \cdot 12 = 2\cdot 64 \text{ sq. ins.}$$

The value of $\frac{q-p}{q'}$ will not be altered by using different factors of safety for p, q and q' and therefore applies to all loads.

These results, arranged for comparison, are collected in the following table.

AUTHOR.	$\frac{q-p}{q'}$	A'	—
		Sq. in.	
Prandtl	·25	3·50	For $p=24000$.
"	·42	5·88	For $p=13571$ from formula $p=16000-70\frac{l}{r}$
Engesser	·32	7·04	For all values of p.
Prichard	·27	3·78	" "
"Modern framed structures"	·12	3·36	For all values of p, straight line formula.
" "	·0535	1·50	For all values of p, Rankine's formula, $c'=18000$.
" "	·0267	·75	For all values of p, Rankine's formula, $c'=36000$.
Keelhoff	·12	2·64	For all values of p.

The following list shows the value of $\frac{q-p}{p'}$, in chord A 9-L adjusted by multiplying the original values by the factors $\frac{2}{\pi}$ and 2 where necessary, for use in the formula

$$A'=k\,A\,\frac{2}{l}\,\frac{2\,r^2}{d}\,\frac{q-p}{q'}$$

Prandtl:	$\frac{q-p}{q'}=$	·25 $p=24{,}000$
"	" =	·42 $p=13{,}571$ from $p=16{,}000-70\frac{l}{r}$
Engesser:	" =	·50 for all values of p
Prichard:	" =	·27 "
'Modern Framed Structures'	" =	·24 for all values of p straight line formula
"	" =	·1070 for all values of p Rankine's formula
"	" =	·0535 " "
Keelhoff:	" =	·19 for all values of p

The practical formulae thus give values for the net section of a lattice bar in chord A 9-L ranging from ·75 sq. ins. to 7·04 sq. ins.

The rule in 'Modern Framed Structures' is capable of giving values ranging from ·75 sq. ins. up to 3·36 sq. ins.

The range of values is even more indefinite than the numerical values indicate, depending as it does on the varying opinions regarding the values to be assigned to the constants of the column formulas.

It is evident that the number of rivets necessary to develop the values of the larger sections given above would make the use of lattice bars impossible. Cover plates and horizontal diaphragms would be required.

The value of $\frac{l}{r}$ for the outer web in chord A 9-L is 44, and for the column as a whole, 34·7. This is not good design as the first value ought to be less than the second.

The latticing between the centre webs is inefficient. Intermediate latticing should have been used on these webs. One of the bars between the centre webs in every panel on the upper face of the chord A. 9-L has a net section of only 1·15 sq. ins. at the centre and 1·5 sq. ins. for a length of about 4 inches, whereas between the centre web and the outside web the section is 2·48 sq. ins.

The bending moments and shears in the webs, the bending moments in the latticing and the compressive stresses in the latticing due to the load on the column have not been considered in this discussion. The theory of the design of latticing has been discussed on the assumption that the curvature in the column under load is negligible, as it ought to be.

When appreciable bending occurs, the total transverse shear is still given by the formula $S=P\ \theta$. On the other hand, when the curvature of the axis of the column varies from point to point, the longitudinal shear S' will not be as great in comparison with S as if the column remained straight, on account of part of the transverse shear being balanced by the resistance to bending of the webs taken individually. Only the difference between these actions is thrown into the latticing and represented by S'.

A method of dealing with the shears which in some respects is simpler than that adopted might have been used. This simpler method is based on the assumption that the small bending stresses at the ends of the webs in a panel of latticing, due to the assumed unity of the column, may be neglected. In this case q will denote the average unit stress in the outer web under eccentric loading and not the extreme unit stress in this web. This method has indeed been partially applied in this appendix. The results do not differ appreciably from those of the general method adopted.

Failure of lower chord A 9-L.

In discussing this failure the original conditions will be assumed to hold, that is $S'=\cdot 54\ S$ and $\cdot 45\ S$ between the inner webs and between the inner and outer webs respectively. It is possible that these two values were closer together owing to the working loose of the latticing between the inner webs.

Assume $P=$ the load at the time of failure $=14{,}000{,}000$ lbs. and $P'=50{,}000$ lbs., a load sufficient, according to experiments made at Philadelphia, to cause slow movement and rivet slip,

$$\text{then } S=\frac{50{,}000}{\cdot 54}=92{,}592$$

$$\text{Now } S=P\ \theta$$

$$\therefore\ \theta=\frac{S}{P}=\frac{92592}{14{,}000{,}000}=\cdot 0066$$

Thus if the obliquity $=\cdot 0066$ existed under a load $P=14{,}000{,}000$ pounds, the chord would gradually go to destruction.

The measurements made by Messrs. Birks, McLure and Kinloch on August 27th, 1907, show when averaged up for the four webs, a deflection of the chord as a whole of $1\frac{3}{4}$ inches at the point between the second and third lattice panels from the south end. Since no measurements were made to determine the position of the axis of the chord from panel point to panel point it is impossible to state the real deflection of the chord, and the only assumption which can be made is that it is represented by the above amount.

It is not possible to state why the maximum deflection took place at the point mentioned. There may have been an original deflection of small amount there, a defect in workmanship or a local injury from the fall mentioned in Appendix No. 11.

The accompanying buckling of chords 8 R and 9 R cantilever arm, shows that the failure itself was not accidental, although it may have been localized by defects or accident. The Philadelphia tests show that slip in the lattice system would have

commenced with an obliquity less than one-half of that obtained above and it is possible that the first movement was caused by a combined stress due to a small initial obliquity and to the shortening of the chord under stress.

It is hardly necessary to note that, owing to the form taken by the chords, partial failure in the lattice system must have preceded any failure in the web system.

If now it be assumed that the chord had an original deflection of ½ inch at the place in question, the inclination of the axis of the chord in the first panel of latticing will be found to be $\frac{\cdot 5}{191} = \cdot 0026$ which is greater than in any other panel. To make up the required obliquity of ·0066 it will therefore be necessary to assume that the direction of the load originally had an obliquity of $\cdot 0066 - \cdot 0026 = \cdot 0040$ to the axis. This would be equivalent to 2¾ inches in the length of the chord, probably due to an eccentricity of about ½ inch to the east at panel point 8-9 and 2¼ inches to the west at panel point 9-10.

From the discussion given in this appendix and from the results obtained in the test of model chord No. 1, it is not difficult to see that failure was certain and close at hand on August 27th. The evidence shows that the increase of obliquity which created this danger condition took place between August 24th and August 27th.

On August 27th the curvature was such that the line of load which would give the least maximum obliquities in the chord had an eccentricity with regard to the centre line adopted for the measurements towards the west of 1¾ inches at panel point 8-9 and ¾ inches to the east at panel point 9-10, equivalent to an inclination of ·004. As this line of load gives minimum lattice stresses we assume it, for the purposes of this investigation, to be the true line of load. The inclination of the axis of the chord in the first panel of latticing at the south end with reference to the same centre line was about ·016, thus making the obliquity of the line of load in this panel about $\cdot 016 - \cdot 004 = \cdot 012$.

The question now occurs how was it possible that the chord could sustain an obliquity of ·012 when an obliquity of ·0066 was sufficient to strain the lattices to the danger point?

If the chord had remained straight, an obliquity of $\cdot 0066 \times \frac{60000}{50000} = \cdot 0079$ would have caused immediate failure.

In reply it may be said, as has already been pointed out, that the consideration of the bending moments in the individual webs accompanying the bending of the chord, the bending moments in the latticing, the compressive stresses in the latticing due to the load on the columns &c., &c., has heretofore been omitted. Of these the first appears to be the most important and its effect in aiding the lattice system may be estimated as follows:—

Let M denote the increase in the bending moments of the outer web in the first panel of latticing at the south end of the chord, M' the corresponding quantity for the inner web,

$$\text{then } 54{\cdot}36\ S' = 69\ S - 2\ (M + M')$$

$$\therefore\ S' = \frac{69\ S - 2\ (M + M')}{54{\cdot}36}$$

the length of this panel being 69 inches.

Now the chord in the length of the first two panels of latticing, viz., in 142 inches has a central deflection of ¼ inch. The radius at the middle point may thus be computed approximately. The resulting value is $r = 10{,}000$ ins. The true radius may be less than this as the web was probably nearly straight next the cover plate with increasing curvature towards the point of greatest deflection. It is even possible that there was a point of contraflexure near the edge of the cover plate.

Now

$$M = \frac{E\ I}{r} = \frac{30{,}000{,}000 \times 366}{10{,}000} = 1{,}098{,}000 \text{ inch pounds}$$

$$M' = \frac{E\ I'}{r} = \frac{30{,}000{,}000 \times 239}{10{,}000} = 717{,}000 \text{ inch pounds}$$

From what has been said with reference to the change of curvature along the length of the chord, it is not unreasonable to assume that the above bending moments represent approximately the increase of bending moment in the length of the first panel of latticing *i.e.,* in 69 inches from south to north.

Thus

$$2\ (M + M') = 2\ (1{,}098{,}000 + 717{,}000) = 3{,}630{,}000$$

$$\therefore\ S' = \frac{69\ S - 3{,}630{,}000}{54 \cdot 36}$$

Now

$$S = P\ \theta = 14{,}000{,}000 \times \cdot 012 = 168{,}000$$

$$\therefore\ S' = \frac{69 \times 168{,}000 - 3{,}630{,}000}{54 \cdot 36} = 146{,}468$$

$$\text{And } P' = \frac{1 \cdot 4}{4} S' = \cdot 35 \times 146{,}468 = 51{,}264 \text{ pds.}$$

Thus on account of the resistance to bending of the individual webs, the obliquity ·012 will produce a stress in the lattice bars of about 51,264 pounds, whereas had the webs remained straight, an obliquity of only ·0079 would have destroyed the lattice bars.

In the tension experiments described in Appendix No. 15 on lattice bars like those used in the Quebec bridge the breaking values of P' were 60,100, 59,800 and 59,500 pounds.

In the above calculations the compressive stresses in the lattice bars due to the compression of the chord as a whole have been neglected.

The above explanation of the failure of this chord under three-quarters of its maximum working load contains assumptions which render it only tentative. It indicates the dangerous effects of even small obliquities and deflections on the safety of a chord with weak latticing. It is quite probable that the obliquity was in great measure due to movements at the field joint in panel 9-L, which was riveted up, and at the field joint in panel 10-L which was being riveted up at the time of the collapse. In fact all the troubles in the lower chords of both anchor and cantilever arms which developed after August 6, 1907, seem to be partly attributable to movement at the field joints. These movements were noticed principally in the inner webs, which have much less horizontal stiffness than the outer webs. These webs were intended to carry the same unit loads as the outer webs, and yet at the field joints they were connected to the cover plates with only half as many rivets, the small web angles used not permitting more. The outer webs with heavy angles and fairly effective latticing seem to have stood up under the stresses—the small angles and inefficient splicing and latticing of the inner webs allowed them to yield, thus disturbing the intended action at the field joints and panel points and giving opportunities for unforeseen eccentricities of loading. Heavier angles on the centre webs under the cover plates, heavier splicing and heavier top and bottom cover plates would have added much to the efficiency of the joints.

An important function of cover plates is that they maintain the webs or ribs at their proper distances apart, but in erection, the bottom cover plate was taken off during the riveting up of the joint, and was replaced by small angle bars which were entirely too slight to perform the function of the cover plate. This is shown by the fact that a much greater movement was noticed at the bottom of the centre webs than at the top.

See Drawings Nos. 25, 26, 27, 28, 29 and 30.

TEST OF MODEL CHORD NO. 1.

Made at Phœnixville by the Phœnix Bridge Company, November 21, 1907.

This chord was essentially a model of chord A 9-L, Quebec bridge, between panel points. It had, however, no field joint. Its dimensions were one-third of those of chord A 9-L. (See Drawing No. 22.) It broke without warning under a load $P = 2,322,600$ lbs., by the failure of the outside lattice rivets.

The ultimate shearing value of one rivet was 4000 lbs. The lattice angle was connected with the web by two rivets.

From the foregoing formulas therefore

$$S = \frac{P'}{k} = \frac{8000}{\cdot 45} = 17,778 \text{ pounds.}$$

$$\theta = \frac{S}{P} = \frac{17,778}{2,322,600} = \cdot 0077$$

The obliquity of the load which caused the failure was thus ·0077, subject to correction for error of calibration of the testing machine.

TEST OF MODEL CHORD NO. 2.

Made at Phœnixville by the Royal Commission, January 18, 1908.

The test of chord No. 1 showed that the lattice system was too light, but gave no indication of the ultimate strength of the column if properly latticed. The capacity of the Phœnix Iron Company's machine was not sufficient to permit a complete test of this kind. In order, therefore, to get results, a chord with only two webs was constructed. The dimensions of the webs were one-third of those of the outer webs of chord A 9-L. (See Drawing No. 23.)

The lattice system, however, was made about twice the strength of that in Model Chord No. 1 and the length of the model was only $11' - 4\frac{1}{4}''$ c. to c. of pinholes. The lattice bars were connected to the web by four rivets instead of two.

This chord fulfilled the expectations of the Commission and broke under a load of 37,000 pounds per square inch by the yielding of the webs in the centre panel.

From what has been said it is evident that this experiment did not settle the question of the strength of the latticing. Stronger latticing might have been required in good design. The proper inference is that the obliquity was too small to break the latticing, so that the full strength of the webs was nearly, if not quite, developed.

Since the inside webs of the Quebec chords are less stiff than the outer webs, it seems to be a fair inference that 37,000 lbs. per square inch is higher than the strength of the Quebec chord would have been, even if properly latticed.

Some allowance also must be made for the higher strength and elastic limit of the small plates and angles used in these models as compared with those in the bridge.

There is doubt as to the correctness of the calibration of the testing machine, so that the above figures are subject to correction. In the tests of both models, the dishing of the webs between the upper and lower lattice systems was small and only careful measurements rendered its existence apparent.

See Drawings Nos. 21, 22, 23 and 24.

In concluding this appendix, some brief comment is necessary upon two points, viz: (1) The use made by Mr. Szlapka of the information existing in 1903 respecting the design of latticing and (2) the application in practice of the theoretical formula given in our discussion.

(1) The use made by Mr. Szlapka of the information existing in 1903 respecting the design of latticing. It has been admitted by Mr. Cooper that he failed to give the design of the lower chords the degree of personal atten-

tion that he gave to the details of the tension system. The foregoing discussion shows that even at the present time theories of lattice design are seriously in conflict and the strength of any lattice system will vary materially according to the formula adopted. Mr. Szlapka used, with his own modifications, the only system of lattice computation generally known to American engineers. This method involved the choice of a column formula from which to determine certain quantities necessary in the lattice computations. Mr. Szlapka selected the column formula adopted by his own company, and used the constants for it that, in his judgment, were most in keeping with the conditions of the case and in best accord with the spirit of the specification. He made what he considered a liberal increase in his adopted sections over what his computations called for. The result has shown that his judgment was faulty, but we are not prepared at this date to define the minimum safe sections for the latticing for these chords. The profession has learned much from Mr. Szlapka's mistake, but it is not yet in a position to determine the percentage of his error. The lattices of model chord No. 2 were proportionately only 50 per cent heavier than those used on the Quebec chords and yet they did not fail until the webs yielded. We have indicated in the discussion that Mr. Szlapka's attention would soon have been drawn to the weakness of the theory by which he was guided, had he made any study of the results given by that theory with different assumptions. No explanation, except the previous uniform success of compression members in service, can be offered for his failure to do this.

(2) The application in practice of the theoretical formulas given in our discussion depends upon our ability to select values of q and p suitable to the detail of construction in the special column under consideration. The values of p are determined in practice by the use of column formulas, but no one contends that the range of the tests upon which these formulas are based is sufficiently extensive to cover all the conditions that affect column strength; the formulas are simply accepted as the best guide that we now have. It is evident that by experience values of q and p may be gradually determined which will make it possible to design latticing that will be unquestionably safe and not unnecessarily heavy. We may here point out that great compression members, such as the Quebec bridge chords, call for just as much individual study in design as an ordinary small bridge, and that any specification for such members should give reasonable latitude for the exercise of judgment by the designing engineer.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY.
J. GALBRAITH.

APPENDIX No. 17.

A COMPARISON OF THE DESIGN FOR CERTAIN CHORDS OF THE QUEBEC BRIDGE WITH THOSE FOR SIMILAR MEMBERS OF OTHER GREAT CANTILEVER BRIDGES, ILLUSTRATED WITH OUTLINE DRAWINGS OF THE BRIDGES AND COPIES OF THE SHOP DRAWINGS OF THE CHORDS.

The outlines of six great cantilever bridges are shown on drawings Nos. 31 and 32 and detail plans of the lower chord construction adopted for each bridge, on drawings Nos. 34, 35 and 36.

The position of the chord selected is shown in each case on the outline drawings, except for the Forth bridge; the detail drawing for this bridge is simply a sketch plan showing the general make-up of the main compression members.

In the attached table we introduce for use in comparison, an example giving the dimensions of an ordinary bridge post of the two channel type, the figures being taken from Professor Burr's 'Elasticity and Resistance of the Materials of Engineering.' These dimensions are more or less typical of those latticed columns that have been used in bridge construction with such success during the last twenty-five years; the details of such columns are now designed entirely by practical rules.

It will be noted that the Forth bridge chord is in a class by itself. It is not a latticed section but may be regarded as a solid section built up out of separate plates. No criticism touching the practical success of this design has ever been made, but it is not a class of construction that could be adopted by an American bridge company without making material changes in its shop equipment and methods of handling its business. We have, however, noted in Appendix No. 18 that the work of the Forth bridge designers is worthy of careful study.

The examples taken from American practice may be divided into three groups:—

(1) Chords of the ordinary two channel type which reaches its maximum development in the Monongahela design.

(2) Chords of the four channel type latticed into one column as adopted for the Memphis and Quebec bridges.

(3) Chords of the four channel type, latticed into two columns which are made to act together by means of tie-plate connections.

This type was adopted for the Thebes and Blackwell's Island bridges.

In the following table we give the principal dimensions of the chords shown on the drawings.

TABLE OF CHORD DETAILS PREPARED FROM DRAWINGS NOS. 34, 35 AND 36.

Bridge.	Area of cross section.	Area of latticing in a section (measured at right angles to lattice axis.)	Area of rivets connecting lattices cut by section to outside web (one end only.)	Length of chord=l.		r in plane parallel to latticing.	l ÷ r.	Weight of plain section per lin. ft.	Weight of latticing per lin. ft. of chord.	Depth of chord back to back of web angles.		Width of chord out to out of web plates.		Section of horizontal splice plates.	Section of vertical splice plates.
	Sq. ins.	Sq. ins.	Sq. ins.	Ft.	Ins.	Ins.		Lbs.	Lbs.	Ft.	Ins.	Ft.	Ins.	Sq. ins.	Sq. ins.
Quebec	781	10	4·8	57	$0\frac{9}{32}$	19·7	35	2,603	66	4′	$6\frac{1}{2}$″	4	$9\frac{3}{8}$	68	212
Memphis	213	10	4·8	28	$2\frac{3}{4}$	14·8	23	710	48	2	$6\frac{1}{4}$	3	2	46	230
Blackwell's Island	852	25	4·8	31	$6\frac{1}{8}$	22·0	17	2,840	144	4	$0\frac{1}{2}$	5	$2\frac{5}{8}$	80	187
Thebes	189	$11\frac{1}{4}$	4·8	30	$6\frac{5}{32}$	20·1	18	630	78	3	$0\frac{1}{2}$	4	$4\frac{3}{4}$	80	70
Monongahela	262	$14\frac{1}{2}$	7·2	30	$6\frac{1}{32}$	25·4	14	873	82	3	$0\frac{3}{4}$	4	1	74	164
"Burr"	35	$3\frac{3}{4}$	1·2	45	0	6·5	83	118	20	1	6	1	$0\frac{3}{4}$	..	....

NOTE.—The horizontal stiffness of the inside ribs of the Quebec bridge chords is less than that of the outside ribs, which is not the case in any of the chords of the other bridges. (See also table in Appendix No. 18.)

It is almost impossible to find any common basis for a comparison of these chords. It must be remembered that latticing is often uniform in size in members on the same bridge doing similar service, but having different loads and cross sections. Thus in the Quebec bridge A 9-L had an area of 781 square inches and A 1-L an area of 301 square inches, yet both members had about the same outside dimensions in cross section, and the same latticing. Therefore as the chords selected for the drawings are not the most heavily stressed chords in the respective bridges, comparison by proportion of lattice to main sections would be unfair. In fact we may say that the drawings given are only typical.

A theoretical comparison between the lattice systems of the different columns might be made by using any one of the various formulas given in Appendix No. 16, but we have already pointed out that no one of these formulas is generally accepted by the profession. There are so many causes of variation in the strength of built up chords of equal area which are not provided for in these formulas that comparison by calculation does not appear to be satisfactory.

Referring to the table it will be noted that the Quebec chord has considerably less horizontal stiffness (see values of $\frac{l}{r}$), less lattice area, less rivet area, and less splice plate area in proportion to the size of the members than any of the earlier bridges. It should be remembered also that the unit stresses for the Quebec bridge were higher than those of the earlier bridges. It will be noted that the earlier designers considerably overran 15 per cent or 20 per cent of splice plate area. This is also true of the Quebec bridge chords, but not to the same extent. Mr. Szlapka states (*see Evidence*) that splice plates having an area of cross section equal to 15 per cent or 20 per cent of the cross section of the member would be satisfactory.

The development of the detail plans of the Blackwell's Island bridge was contemporaneous with that of the Quebec bridge plans; the Quebec designers had not access to the Blackwell's Island plans. In fairness to the Quebec bridge designers, however, it should be pointed out that in the Blackwell's Island bridge the proportions of many of the details are much more nearly in accord with Quebec bridge practice than are those of the earlier bridges, although the principles of the designs are very different.

A consideration of the difference in the designs on drawings Nos. 34, 35 and 36, all of which have been prepared under the direction of engineers of recognized ability and high professional standing, shows that there is as yet no established system of design for large compression members. The individual judgment of the engineer is the determining factor, and this may prove to be erroneous as it did in the case of the Quebec bridge.

The lack of precise knowledge on this subject has been discussed in other appendices.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY,
J. GALBRAITH.

APPENDIX No. 18.

A CRITICAL DISCUSSION OF CERTAIN PARTS OF THE SPECIFICATIONS.

The Quebec bridge was designed to meet the requirements of the specifications approved by the Dominion government in 1898 and amended in 1903. The method adopted by the company to procure tenders was to issue a general specification and to call upon contractors to prepare plans in accordance therewith.

Considering all the conditions pertaining to the undertaking the adoption of this method was not in the best interests of the work. The company was known not to have the capital necessary to immediately proceed with construction, and the preparation of complete preliminary plans would involve a large outlay. The evidence and documents show that the preliminary plans submitted with the tenders were incomplete; this was as might have been expected, as the several contractors who tendered for the work had little assurance that they would get any return for their expenditure of time and money.

Specifications as a rule consist of two distinct portions, one of which relates to design and the other to fabrication, material and execution. In the case of the Quebec bridge, the difficulty of preparing an adequate specification for design was very great. It would have been better to have entrusted the preparation of the plans and specifications to engineers independent of any contracting or manufacturing company, whose previous experience qualified them to handle the work. This course would have avoided duplication of designs involving expensive plans and would have prevented the letting of a contract on incomplete plans formed upon erroneous data; the engineers would have made a proper and sufficient study of the whole project, and in due time competitive tenders upon their plans would have been secured, thus enabling all contractors to tender on a common basis. The privilege of submitting independent plans might have been extended to the bidders. The reason for not following this course is explained by Mr. Hoare in his evidence.

The procedure as outlined above would have been applicable to an enterprise which involved so many new problems and the application of existing knowledge on so large a scale and which demanded the continual exercise of sound judgment.

An error of judgment made by the Quebec Bridge Company was the selecting of an engineer who did not possess the necessary special knowledge and experience to prepare the specification (see Appendix No. 7). It is true that this specification was considered to be only tentative, drawn up for the purpose of procuring preliminary tenders, but its history and importance cannot be overlooked. (See Appendix No. 6.) It became the basis of the contracts between the Quebec Bridge Company and its contractors, was approved by the government engineers, and was an essential part of the subsidy agreement whereby the Dominion government undertook to pay the Quebec Bridge Company on certain conditions, one million dollars (Exhibit 12).

The specification itself (Exhibit No. 21), herein called the 1898 specification, was for the most part a copy of a specification issued by the Department of Railways and Canals in 1896; there is nothing in its wording to indicate that the Quebec bridge was an exceptional structure and without precedent or that the propriety of applying to this structure other than the usual clauses in bridge specifications was carefully considered.

We do not think that any engineer would be justified in writing a specification without consulting freely those specifications most used in practice; specifications are in fact the statement of the provisions that engineers have in the past been forced to make in order to secure satisfactory results, and each succeeding revision is the outcome of experience. Such compilations are necessary and cannot be dispensed with, but this fact does not justify an engineer whose special experience has not fitted him to judge of the importance of vital clauses, in revising and rearranging them. The danger in so doing lies in the fact that a clause necessary and useful in one specification may not be applicable under other conditions, and opinions on such matters are valuable only from men of special qualifications. Errors arising from the compilation of specifications by experienced men are by no means uncommon. Mr. Cooper recognized this and so revised the specifications of 1898.

In regular bridge practice the specification is of importance particularly because an American bridge works is a factory for turning out structural steel fabricated in accordance with plans prepared in the drawing office attached to the works. This drawing office is a part of the factory, and in it, as throughout, efficiency is obtained by standardizing and duplication; the drawing office staff consists of a number of well trained computers and draughtsmen whose duty it is to prepare the shop drawings for the work and who are under the control and direction of a designing engineer. Details are designed in accordance with the specifications furnished by the purchaser, except under circumstances when shop equipment requires some deviation to be made to secure facility of manufacture. It is not a part of the duty of the drawing office staff to question the wisdom of the requirements of the specification, nor could the progress of work throughout the factory be satisfactorily maintained if it should attempt to do so.

The evidence shows that the Phœnix Bridge Company followed this usual practice in the preparation of the Quebec bridge designs.

In 1903 it became necessary to design the main spans of the bridge and the 1898 specification was amended by Mr. Cooper, it having been understood ever since 1900 that it would be amended and altered. The history relating to the adoption of these amendments is given in Appendices Nos. 3 and 6.

Mr. Cooper did not recognize these amendments as complete and final, and considered that he had the power to deal with each problem of design as it arose, and he exercised this power when he thought it necessary. The designing of the main span was left to Mr. Szlapka, Mr. Cooper having approved the specifications and no one questioned any decision that these engineers made. The work was done under the immediate direction of Mr. Szlapka.

Before discussing the specification, it will be well to contrast some of the main features of the Quebec bridge with those of other cantilever bridges and the following table is inserted for this purpose:—

INFORMATION CONCERNING GREAT CANTILEVER BRIDGES.

INFORMATION CONCERNING

Name.	Date of Construction.	Designer.	Contractor for Superstructure.	Span.	Width C. to C. Trusses.	Live Load per Lin. Foot.
				Feet.	Feet.	Lbs.
Forth	1882–1889	Baker & Fowler...	Wm. Arrol & Co..	1,710	Varying, lower chord $31\frac{1}{2}$ at ends to 120 at piers.	Double track Ry. 2,240 lbs. per track.
Memphis.....	1886–1892	Geo. S. Morrison .	Union Bridge Co..	790	30	Single track Ry. 4,000 lbs. per track.
Monongahela .	1902–1903	Boller & Hodge...	American Bridge Co.	812	32	Double track Ry. 4,500 lbs. per track.
Thebes	1902–1905	Noble & Modjeski.	American Bridge Co.	671	32	Double track Ry. 5,000 lbs. per track, less 20 p.c.
Blackwell's Island	1901–1908	Dept. Bridges New York City.	Pennsylvania Steel Co.	1,182	60	Roadway and trolley ordinary 8,000 lbs. congested 16,000 lbs.
Quebec	1900	Phœnix Bridge Co.	Phoenix Bridge Co..	1,800	67	Double track Ry., roadway and trolley 4,000 lbs. per track. For extreme conditions mult. $1\frac{1}{2}$.

GREAT CANTILEVER BRIDGES.

Ultimate Strength for Steel Unit 1,000 lbs.	Approx. Weight of Steel in Short Tons per Lin. Ft.	Price of Structural Steel per Lb. Cts.	Working Stresses—Lbs. per sq. inch.	Allowed Shear on Rivets—Lbs. per sq. inch.
Compression, 76-83. Tension, 67-74.	10½	6·50	Max. stresses, Compression, 17,000. Tension......16,350	About 12,000
Compression, 69-78½. Tension, 66-75.	3½	5·88	Compression, 14,000, if $l < 16d$,—deduct 750 lbs. for each additional unit over 16 in $\frac{l}{d}$; tension for dead load, 20,000; tension live load, 10,000.	7,500
Compression, 60-70. Tension, 63-75.	4½	4·3	Compression dead load, 21,000 where $\frac{l}{r} < 40$. Tension dead load, 22,000. Take one-half in each case for live load.	10,000
62-72.	5	*5¼	Compression dead load, 21,000 if $\frac{l}{d} < 16$. Tension, 20,000. Take ½ in each case for live load.	7,500
Compression, 60±4. Tension, 66±4. Nickel steel eyebars, 85.	13½	*5⅝	Compression, ordinary, $20{,}000 - \frac{l}{r}$; congested, $24{,}000 - 100\frac{l}{r}$ Tension, ordinary, 20,000; congested, 24,000. Tension for nickel steel, ordinary, 30,000; congested, 39,000.	Ordinary, 13,000 Congested, 16,000
Compression, 60-70. Tension, 62-70.	13	5·60	Compression, ordinary, $12{,}000\ (1+\frac{\text{Min.}}{\text{Max.}})$; extreme, 24,000; both for $\frac{l}{r} < 50$. Tension, ordinary, $12{,}000\ (1+\frac{\text{Min.}}{\text{Max.}}$ Extreme, 24,000.	¾ working stress =18,000 extreme.

* Not official.

It is not possible to set forth all the facts in a table with sufficient minuteness to justify the making of complete comparisons, as many qualifying clauses and special conditions are necessarily omitted. Three items of interest may be noted in the table:—

(1) Only the Forth bridge is at all comparable with the Quebec bridge in regard to span.

(2) Only the Blackwell's Island bridge is comparable with the Quebec bridge in regard to unit stresses both for main members and for details.

(3) All the bridges included in the table were designed by independent engineers except the Quebec bridge.

In this connction we must express the opinion that it is difficult for the employees of a large manufacturing concern to give the design of a bridge of unique features the concentrated attention that it requires.

With regard to precedent only the Forth and Blackwell's Island bridges involved anything like the same total stresses as the Quebec bridge. The design and construction of the Blackwell's Island bridge was contemporaneous with the Quebec bridge.

The Forth bridge was built on a system not suited to the established American methods of bridge construction, so that its distinctive features of design, construction and erection were not followed. It is proper to add that the achievements of the Forth bridge engineers deserve much closer study than appears to have been given to them on this continent. Messrs. Baker and Fowler succeeded in erecting a structure which weighs considerably less per lineal foot than the Quebec bridge and which is designed to carry about one-half the rolling load and several times the wind load specified for the Quebec bridge. The main compression chords of the two bridges are of practically equal area, but the material in the Forth bridge is of a considerably higher ultimate strength than that used in the Quebec bridge, the unit stresses are less and the design of the cross section of the chords is such that they should be able to carry a greater unit stress with safety. On great bridges these are factors to be observed and it is to be regretted that the stress sheets and full engineering studies in connection with the Forth bridge have not been published.

It is evident that the designers of the Quebec bridge were compelled to work from experience gained on much smaller bridges.

In discussing the specification we deal not only with the clauses immediately connected with the downfall, but with others that were not in our judgment calculated to ensure a safe and satisfactory structure.

The specification is here understood to mean the 1898 specification as amended in writing by Mr. Cooper.

As a document the specification is unsatisfactory, some of the clauses having been amended by Mr. Cooper, some set aside in favour of his well known and generally accepted standard specification and some remaining in force with a context that altered their meaning. No general or complete revision of the specification embodying Mr. Cooper's amendments was ever compiled.

As a matter of fact although the 1898 specification was retained as the official specification and much of the work done in accordance with it, we believe that Mr. Cooper depended upon his own inspection of the plans under the revised specifications to secure satisfactory details. His opinions upon most debatable questions of design were well known to the staff of the Phœnix Bridge Company, which had previously designed and built many structures under his direction and was accustomed to his methods. It is on record that the Phœnix Bridge Company requested Mr. Cooper to set aside the 1898 specifications altogether and to substitute for them his own standard specifications.

A complete bridge specification must set forth the character of the material that is to be used, the loadings that are to be carried, the stresses to be permitted in the members and provisions concerning details, fabrication and erection to be observed; in fact everything essential to the proper carrying out of the work as intended.

The 1898 specification for material was used without alteration except in one particular, Mr. Cooper having raised the minimum limit for the ultimate strength of eyebar material from 60,000 lbs. to 62,000. The metal specified was the ordinary grade of structural steel.

It will be noted by reference to the table that the Quebec bridge specification called for material of slightly lower ultimate strength than that used in any of the other bridges while the bridge itself had the longest span of all. The need of a better material than structural steel for the construction of long span bridges is generally recognized, because the decrease of total weight and consequently of cost in a large truss with increase of permissible unit stress is very rapid. In the Quebec bridge the dead load stresses constituted roughly two-thirds of the stress on the main members.

The designers of the other two great bridges introduced special grades of steel so that high unit stresses could be safely used. The Forth bridges engineers were not permitted to load their metal to more than one-fourth its tensile strength, and for compression, used a steel of about 25 per cent stronger than that supplied for Quebec. Nickel steel with a permissible unit stress 50 per cent higher than allowed on material in the same bridge and similar to that used at Quebec was introduced into bridge practice by the Blackwell's Island bridge engineers. The use of this alloy as a structural material was investigated and favourably reported upon in 1903 by a special commission of which Mr. Cooper was a member.

It was Mr. Cooper's opinion that it was wiser to use the ordinary grade of metal for the Quebec bridge and to load it to the highest working stresses that were considered practically safe.

ELASTIC LIMIT.

We do not know whether Mr. Cooper in his amendments intended the term 'elastic limit' to mean the elastic limit of a test specimen or of a full sized member. There is also some uncertainty as to the true meaning of the term 'elastic limit,' which is unfortunate as the maximum working stresses specified are made to depend upon this characteristic of the material.

The 'elastic limit' accepted by bridge designers as a controlling factor in their work cannot be determined by the method prescribed by the 1898 specification, and yet this method (the drop of the beam) was used. Both Mr. Cooper in his standard specifications and the engineers for the Blackwell's Island bridge provide for a much closer determination of this characteristic. In reality the determination is a delicate and time consuming process for a research laboratory and impossible under the conditions existing in a rolling-mill; to such an extent is this true that it is not called for in the carefully prepared specification issued by the American Railway Engineering and Maintenance of Way Association in 1906. The principle apparently followed in the latter specification is that mill tests are sufficient for mill purposes and that the true elastic limit can be most safely obtained by proportion from the ultimate strength. The assumption generally made is that the true elastic limit for structural steel is about 50 per cent of the ultimate strength.

The material actually supplied for the bridge was regularly tested and a comparison between its probable elastic limit and the 32,000 lbs. per sq. in. apparently expected by Mr. Cooper, is possible. The full size eyebar tests, a record of which will be found in Exhibit 86, show that the metal in service shape had a safe ultimate strength not in excess of 55,000 lbs. per sq. in. and a reported elastic limit of 28,000 lbs. per sq. in. These tests were made on long bars in the Phœnix Iron Company's large testing machine and the results might be reduced by calibration of the machine and closer observation of the elastic limit. It will be noticed that the proposed extreme working stresses (24,000 lbs. per sq. in for the Quebec bridge were nearly equal to the elastic limit of the eyebars.

The elastic limit in compression was assumed in accordance with the usual practice to be the same as that in tension. An examination of the voluminous test

records (Ex. No. 28) shows that an ultimate strength in excess of 60,000 lbs. per sq. in. was not regularly secured, so that, accepting the 50 per cent relation mentioned above, the elastic limit in compression becomes 30,000 lbs. per sq. in. It should be noted that these tests were made on specimens of about one-half of one sq. in. sectional areas. The compression members were built up of wide thin plates riveted together into webs. We know of no test that has ever been made to establish the relation between the strength and elastic limit of such plates and those of small test specimens, nor do we know what effect the punching, riveting and painting have on the material in the webs as compared with the solid plate. It was noted at the wreck that the paint between the plates of members that had been fabricated for over three years was still fluid. From the analysis of full-sized tension tests we think it possible that the elastic limit of the plates in the compression members was not much above 27,000 lbs. per sq. in. instead of 32,000 lbs. as apparently assumed.

UNIT STRESSES.

The maximum unit stresses that Mr. Cooper proposed to use were about 21,000 lbs. per sq. in. under ordinary loading and 24,000 lbs. per sq. in. under extreme conditions. He considered that the extreme conditions as specified would never occur.

By reference to the table it will be seen that the specified stresses for the Quebec bridge under working conditions are in advance of currnt practice and we believe that they are without precedent in the history of bridge engineering. Under extreme conditions the Quebec bridge stresses are in general harmony with those permitted in the Blackwell's Island bridge.

We have already indicated that the dimensions of the Quebec bridge were such that the use of the highest safe unit stresses was justifiable and good engineering practice. If we were sure that the loads were correctly estimated, that the stresses acted in the bridge exactly in accordance with the assumptions and that the elastic limit of the built-up members was not less than 32,000 lbs. per sq. in, 24,000 lbs. per sq. in would not be an unsafe stress for structural steel, provided that the material is regular in quality and the details satisfactorily worked out to suit such a stress.

Mr. Cooper provided for the effect of live load by the use of the so-called fatigue or $\frac{\text{min}}{\text{max}}$ formula. This method which was formerly much used has more recently been abandoned in general practice and is not adopted by Mr. Cooper in his standard specifications. In the hands of an experienced engineer this method will be made to produce much the same results as the more modern impact formulas. We do not know why this formula was used in this case, except that it was adopted by Mr. Hoare in 1898 from the 1896 specifications of the Department of Railways and Canals and was probably retained in 1903 for convenience.

Mr. Cooper adopted the ordinary straight line formula for compression members making the dead load unit stress equal to $(24{,}000 - 100 \frac{l}{r})$ lbs. per sq. in. We have already indicated in Appendix No. 13 that this formula is purely empirical and does not agree particularly well with the recorded tests upon large columns. It is the most generally accepted formula of practice, but we do not believe that the engineering profession has at present a satisfactory knowledge of the action of large steel columns.

There is a wide field for experiment which must be worked over before engineers can claim to have a sufficient knowledge of steel to design both safely and economically, and perhaps the most serious criticism of the structural engineers of the present day is that they have permitted this field to remain undeveloped for twenty-five years' durduring which time they have adopted a new metal for their work and new shapes and sections.

We think that in popular engineering opinion the ultimate strength of steel columns is largely over-estimated, the diagram on drawing No. 20 indicating that for

the Quebec chords it was not safe to expect an ultimate strength in excess of 32,000 lbs. per sq. in, so that under the extreme conditions specified the margin of safety would only have been one-third.

This is a point on which current engineering practice is open to direct criticism. The older engineers, upon the results of whose experiments the profession is now depending, did not think of loading metal in compression to the unit stress used in tension because they recognized that the ultimate unit strength of members in compression was far less than that of members in tension.

The later school of engineers seems to have adopted the principle that the action of bridge members under stresses in excess of the elastic limit is a matter of indifference as they will never be so stressed. The action within the elastic limit being practically the same under both conditions, they adopt the same working stresses in tension and in compression. Their practice has been attended with complete success, but this may be attributed to the fact that the material has ordinarily not been stressed to much above half the elastic limit.

Under the Quebec bridge conditions, where high working stresses were imperative, the wisdom of the practice of loading in compression as heavily as in tension becomes questionable. We believe that in no great public structure should stresses be permitted in excess of one-half of the ultimate strength of any compression member, no matter how high the elastic limit may be.

It will be noted that Mr. Cooper in specifying the stresses for the lower chords of the Quebec bridge omitted the term in the column formula containing the ratio $\frac{l}{r}$. In this practice he is supported by the engineers of the Monongahela and Thebes bridges, who made a similar provision, but reduced the maximum stress to that allowed by the usual formula for a column with $\frac{l}{r}$ equal about 40.

The failure of the Quebec chords does not prove that Mr. Cooper was theoretically incorrect and cannot be directly connected with this clause in the specification. The specification, however, permitted stresses in advance of any previous practice and the proportioning of columns to safely carry such stresses is yet to be learned.

We have already pointed out the seriousness of the error made in the estimation of the dead load which resulted in computed stresses nearly 10 per cent higher (see Evidence) than had been expected. A comparison of these computed stresses with the elastic limit of the material as estimated from the test records will show how narrow a margin of safety was provided in the actual design.

We are not prepared in the present status of the art of bridge building to approve the unit stresses stated in the amended specification.

RIVET STRESSES.

It will be noted from the table that the rivet stresses used were much in excess of previous practice. These seem to have been adopted almost by an oversight. The 1898 specification contained a clause usual in low stress specifications, permitting the rivets to be worked to three quarters of the allowed stress in the member. This clause was not cancelled by the 1903 amendments and under extreme conditions permitted a stress in rivet shear of 18,000 lbs. per sq. in. The tests made in 1904 under the direction of the American Railway Engineering and Maintenance of Way Association have established the fact that a riveted connection begins to work under a stress in rivet shear betweeen 12,000 and 15,000 lbs. per sq. in. and that deformation in even a simple connection is marked when a stress of 25,000 lbs. per sq. in, is reached. These results have been confirmed both in tension and compression by the tests made for the commissioners (see Appendix No. 15). It is therefore clear that the Quebec specification permitted the use of stresses in details which were outside the limits of established

practice and are now know to be unsafe. Knowledge of the action of the rivets in riveted connections is very incomplete.

BUILT UP COLUMNS.

In our findings we have stated that the bridge failed through weakness of the lower chords and particularly in the latticing of those chords. In Appendix No. 16 will be found a discussion of lattice design and of the data that Mr. Szlapka had to guide him in his work. The main outline of the latticing in the Quebec bridge was sketched as early as 1898. There was practically nothing in the specification that was of any service to the designers in this connection and they violated none of its provisions in the design. There are some clauses dealing with latticing, but they were copied from small bridge practice and were wholly inadequate for the Quebec structure. The main criticism that can be made of the designers was that they had the means of checking their theories by use of the testing machine and that they did not do this nor did they thoroughly study the possibilities of lattice formulas.

LOADINGS.

In 1903 Mr. Cooper revised the loadings, increasing the specified train loads and decreasing the wind pressures. While Mr. Cooper undoubtedly made an improvement on the 1898 specification in this respect, he does not seem to have taken full advantage of the improved financial situation due to the decision of the government to guarantee the Quebec Bridge Company's securities. This is explained by Mr. Cooper in his evidence in which there is no reference to the changed financial conditions. (See Appendix No. 5.) Mr. Cooper apparently did not realize the great change in the traffic conditions that would probably follow the opening of the National Trancontinental Railway nor the demands for transportation resulting from the rapid development of Canada. His specified train loading is not greater than that used regularly in Canadian practice and is lighter than that subsequently adopted for the National Transcontinental Railway, and sufficient provision was not made for probable increases of live load.

Considering together the high unit stresses permitted and the loads specified, the specification was not for a bridge well suited to the purposes it would have been called upon to serve.

DEAD LOAD.

The specification requires that the dead load used for calculating the stresses shall not be less than the actual weight of the structure when completed. The evidence shows that the designers failed to comply with this requirement. The effect of their error is shown on drawing No. 4 (see also exhibits 98, 100 and 101). In view of the high unit stresses specified this error was serious enough to have required the condemnation of the bridge even if it had not failed from errors in the design of the compression chords.

The obvious intention of the clause was to compel the designers to check their assumed dead loads by actual calculations from their detailed drawings as soon as these were developed, and it carried with it an obligation on the consulting engineer not to approve any drawings until he was satisfied that the assumed weights were ample. It is not customary in practice to be exacting about the observation of this clause because the weight of an ordinary span for a given loading can be very closely estimated; but no excuse can be offered for applying the precendents of practice to a structure that was entirely outside the range of experience. No evidence has been given to show that any effort was made either in the Phœnix Bridge Company's office or by the consulting engineer to check the assumed weights at the earliest possible date and the error was passed without notice until a large portion of the bridge had

been actually built in the shops and the members weighed. It is in evidence, and we have already stated, that the scale weights were within 1 per cent of the weights as finally computed from the drawings. The consequences of this error were considered by Mr. Szlapka and Mr. Cooper before erection was resumed in 1906 and they state that it was their opinion that the error was not fatal to the safety of the bridge, and the work of erection was proceeded with.

ERECTION.

No special provision is made in the specification for an oversight of the methods of erection by the Quebec Bridge and Railway Company's engineer, or for his approval of the general system of erection, or of the means adopted to solve the various problems arising in connection with it. There is no evidence to show that anyone outside the Phœnix Bridge Company attempted to deal with this practical problem. Mr. Cooper states that the erection plans and devices were not subject to his approval although he was advised of them unofficially and general progress on erection was regularly reported to him.

It was apparently intended, as is the usual practice, to leave all such arrangements in the hands of the contractor, making him provide all necessary plant and holding him responsible for everything that might happen.

The erection staff of a large construction company is best qualified by experience to design erection plant. We are of opinion, however, that the erection difficulties to be met with on a structure like the Quebec bridge are so serious and the necessary risks to be run during erection are so great that if the employment of a bridge engineer is necessary at all, it is especially necessary in this connection. In fact the responsible engineer on such a project should direct the work in all its branches and the contractor is entitled to look to him as a trained specialist for instructions and assistance at all times and especially in emergencies.

The specification throughout shows that the whole subject was not considered with sufficient care not only from a technical standpoint but from the practical or business standpoint as well. Inconsistencies are of frequent occurrence; ambiguity and lack of precise definition pervade the whole, and we desire to direct particular attention in this connection to the important clauses 4, 5, 6, which read as follows:—

(4) After the stress sheets have been approved and before the construction of any part of the structure shall be proceeded with, complete working drawings shall be furnished, showing all details of construction, which shall conform to the general design, shapes and dimensions shown on the stress sheets and to the conditions of this specification. The drawings shall be approved by the engineer before the work of construction is proceeded with.

DRAWINGS.

(5) After the final detail drawings referred to have been approved by the engineer, the contractor is to prepare his shop drawings from the detail drawings, complying carefully therewith, and making no changes without the written consent of the engineer. Working drawings are to be sent in triplicate for the approval of the engineer, who will retain two sets and return the third after making thereon any corrections required, after which the required number of corrected sets will be sent by the contractor to the engineer without delay. The approval of the said working drawings will not relieve the contractor from the responsibility of any errors thereon.

(6) The requisite number of copies of general and detail drawings for all purposes shall be furnished by the contractor upon orders of the engineer.

HENRY HOLGATE,
Chairman.

J. G. G. KERRY,
J. GALBRAITH.

APPENDIX No. 19.

MISCELLANEOUS—QUEBEC BRIDGE INQUIRY.

WEATHER CONDITIONS.

The temperatures and wind velocities for some weeks preceding the accident are shown on drawing No. 37. It will be noted that there were no exceptional conditions in either case, both temperature and wind being moderate and usual. The wind blowing at the time of the accident was so light that wind pressure has not been included in calculating the stresses existing at that time. The drawing shows a wind velocity late in the day on August 29, of about 25 miles per hour, which would theoretically produce the almost negligible pressure of about 2 lbs. per sq. ft., on the truss surface exposed. The form of the truss is such that a correct analysis of the wind forces is most difficult to make and it was considered that less error would result from the neglecting of these forces than from an effort to determine them accurately.

A list of the maximum wind velocity recorded at the Quebec observatory is given on drawing No. 37. This list indicates that the pressure of 25 lbs. per sq. ft. assumed in the 1898 specifications was sufficient for the site, a wind velocity of nearly 90 miles per hour being necessary to produce such a pressure.

The following record of deflections, which is filed as Exhibit No. 55 is of interest as furnishing data for predicting the movements of cantilever arms under wind.

OBSERVATIONS ON THE DEFLECTION OF THE CANTILEVER ARM UNDER HEAVY WINDS.

November 12, 1906.—Front leg of large traveller at P-1. Panel 2 of cantilever arm partly erected. East wind 55 miles an hour. Deflection taken on middle of first transverse strut above deck between posts P-1.

Deflection observed—2½ inches.

November 16, 1906.—Front leg of large traveller at P-1. Panel 2 of cantilever arm almost completed. East wind, 65 miles an hour. Deflection taken at same point.

Deflection observed—3½ inches.

February 3, 1907.—Front leg of large traveller at T O cantilever arm erected complete. West wind, 45 miles an hour. Deflection taken at same point.

Deflection observed—2 inches.

HENRY HOLGATE,
Chairman.
J. G. G. KERRY,
J. GALBRAITH.

REPORT

ON

DESIGN OF QUEBEC BRIDGE.

By C. C. SCHNEIDER.

PENNSYLVANIA BUILDING,
PHILADELPHIA, PA., January, 1908.

SIR,—By telegram of September 9, the writer was appointed by you, on behalf of the Dominion government, with the approval of the Honourable the Minister of Railways and Canals, for the following purposes:—

'To inquire into and pass upon the sufficiency of the present design of the Quebec bridge, which collapsed on the 29th of August, 1907; to thoroughly examine the plans of the superstructure and members thereof, &c.; to look thoroughly into all matters in connection witth the proposed reconstruction of the said bridge, and to state whether, in his opinion, the present design is sufficient.'

After receiving your verbal instructions, the writer visited the site of the Quebec bridge in order to examine the collapsed structure; and immediately commenced to collect such information as might aid him in his work, and proceeded with the examination of the plans, which he received from your department, September 17, 1907.

Not being limited in the scope of his investigations, he understands his duty to be to report on the following questions:—

First.—The sufficiency of the present plans of the Quebec bridge, as to their conformity to the specifications as approved by the government.

Second.—The advisability of modifications in the present plans, should they be found inadequate, using as far as practicable the fabricated material now on hand.

Third.—The advisability of discarding the present plans of the Quebec bridge, and recommendations as to a new design.

The writer has thoroughly investigated the subject submitted to him, and now has the honour to submit the following report:—

The present design of the Quebec bridge is a cantilever of 1,800-feet span between centres of piers, with a suspended span of 675 feet, two cantilever arms each 562 feet 6 inches long, and two anchor arms each 500 feet long; making a total length of 2,800 feet, not including the approach spans, which will not be considered in this report. The transverse distance between centres of trusses is 67 feet. The bridge is to carry two steam railway tracks and a roadway on each side 17 feet wide in the clear, suitable for ordinary highway traffic, with one electric railway track on each roadway.

The writer has computed the strains resulting from the loads given in the specifications as revised by Mr. Theodore Cooper, March 2, 1904, a copy of which is attached to this report in Appendix A.

In comparing the results of his computations with the strain diagrams submitted by the Phœnix Bridge Company, he has come to the following conclusions:—

Floor system.—The sections required in the floor-beams and stringers conform to those required by the specifications.

Trusses.—The strains in the trusses resulting from the live load agree with those computed by the writer. The strains from the dead load as computed by the writer, however, are greater than those shown on the diagram submitted by the Phœnix Bridge Company, for the reason that the actual weight of the steel superstructure is in excess of that estimated previous to its construction.

Bracing.—The strains and sections of the various members composing the lateral and sway bracing of the trusses and the bracing of the floor system as well as their details and connections are in accordance with the requirements of the specifications.

Appendix B accompanying this report gives the writer's computations of the strains in the main members of the trusses. The strains resulting from the dead load are based on the actual weight of the structure taken from the shipping weights of the steel work and distributed in accordance with the positions of the various members, thus representing the conditions which would exist in the finished structure. These loads as concentrated on the various panel points of the trusses are shown on diagram included in Appendix B. The table also contains the sectional areas of the members as shown in the shop drawings, the unit strains required by the specifications and the unit strains as they would occur in the completed structure, based on the actual weight of the members; also strains occurring during erection under conditions existing August 29, 1907.

The tables in Appendix B have been computed in accordance with the writer's interpretations of the specifications, which are:

That the value of $\frac{\text{max.}}{\text{min.}}$ by which the permissible unit strains are determined is derived from the dead and live loads only; but that in proportioning the members these unit strains shall be used for the sum of the strains from dead, live and snow loads.

That as the specifications require that 'only $\frac{1}{3}$ of the maximum wind force need be considered in proportioning the chords,' and nothing is mentioned in reference to the web system, this also applies to wind strains in web members.

That in the formulæ under the head of 'Combined and reversed strains,' L_1 denotes the live load strain of opposite sign from that of the dead load; that the expression '$D - L_1$' is the arithmetical difference between these strains; and that '$D + L + L_1$' is the arithmetical sum of these strains.

By examination of this table, it will be noticed that the actual unit strains in most of the members of the trusses exceed the limits of the specifications. In the upper chords of the cantilever arm (excepting in the panels from U_2 to U_6, which were proportioned for the erection strains), from 10 to 18 per cent; and in the lower chords (with the exception of the panels from L_0 to L_4, which were also proportioned for the erection strains), from 7·5 to 24 per cent. In the upper and lower chords of the anchor arms, the unit strains in all panels exceed these limits from 11 to 20 per cent. The unit strains in the chords of the suspended span also exceed the limits of the specifications: the upper chords 16 to 18 per cent; the lower chords from $7\frac{1}{2}$ to $9\frac{1}{2}$ per cent. While the strains in some web members come within the limits, in some cases they are in excess as much as 21 per cent, and in one case 57 per cent. The trusses, therefore, as designed, do not conform in this respect to the requirements of the specifications approved by the government.

However, there are other points affecting the strength of the structure, not covered by the requirements of the specifications, to which the writer begs to call your attention. These refer more particularly to certain details which appear to have been left to the judgment of the designer.

The writer considers the details the most important parts of the design of a permanent structure, even more so than the general proportions of its members.

Most of the details and connections have received careful and conscientious consideration, and are generally in proportion to the members which they connect and in accordance with the standards of good practice. However, there is a deficiency in many of the compression members, as their connections—such as the latticing—are not sufficient to make the parts composing them act as a unit. The most pronounced defect in this respect exists in the lower chord members of the cantilever and anchor arms. These members consist of four separate ribs, not particularly well developed as compression members, and their connections to each other are not of sufficient strength to make them act as a unit.

As discussions on this subject have of late appeared in print, asserting that a scientific method of proportioning the latticing of compression members is not known, the writer takes exception to these statements, and claims that the strains in lattice bars can be computed with enough accuracy to make them sufficient to develop the full strength of the member.

A discussion on the theory and strength of compression members, including an analysis of the strains in lattice bars, will be found in Appendix C accompanying this report.

DISCUSSION OF PERMISSIBLE UNIT STRAINS.

As the present design of the trusses of the Quebec bridge does not conform in all respects to the requirements of the approved specifications, the question arises: Are the trusses as designed strong enough to carry the specified loads without considering the specifications?

In order to decide that question it is necessary to consider the maximum unit strains which might be permitted in the members of the trusses as coming within the limits of safety. If we knew all the strains occurring in a member of a structure, and if the material and workmanship were perfect, we could allow strains up to the true elastic limit of the material. These ideal conditions of material and workmanship, however, cannot be realized in practice, and in addition to the computed direct strains on which the proportions of the members are based, there are secondary strains produced by the bending from their own weight and deformation of the trusses under load. Allowance must, therefore, be made for these contingencies in determining on unit strains which may be considered within the limits of safety.

The specifications provide for two kinds of live loads for the trusses:—

First. A live load consisting of a train on each track. The strains produced by this load, together with the dead load and specified snow load, are limited to a certain unit strain per square inch.

Second. A provision for future increase of 50 per cent in the live load. For the strains produced by this extreme live load, together with the dead and specified snow loads combined with the wind force, a higher unit strain is specified.

The first case will be called hereafter the working load, and the second case the extreme load. The strains produced by the working load, which is by no means excessive, should leave a reasonable margin for safety. The strains produced by the extreme loads should remain within the elastic limit of the material.

Tension Members:

Eyebars.—The elastic limit in full-sized annealed eyebars cannot be depended upon to be more than 28,000 pounds per square inch. A direct tension of 24,000 pounds per square inch, together with secondary strains caused by the friction on the pins during deformation, and the uncertainty of a uniform distribution of the strains over all the bars, may increase the strain to at least 27,000 pounds per square inch, which is just within the elastic limit, with practically no margin for safety.

A strain of 21,000 pounds per square inch in direct tension combined with the secondary strains, &c., may produce an extreme fibre strain of about 24,000 pounds

per square inch, or 6/7 of the elastic limit of the eyebars. The unit strains to be allowed on eyebars for direct tension, therefore, should not exceed 24,000 pounds per square inch for the extreme load.

Compression Members:—

In accordance with the accepted theory of compression members, the fibre strain near the center of a column increases in proportion of the length to the least radius of gyration, and, therefore, an allowance must be made for the buckling caused by the tendency to bend.

The usual practice in bridges of ordinary span is to consider the gross section of the compression members in computing their strength. This is generally done in connection with the conservative unit strains of about half of the elastic limit, thus giving a considerable margin for safety; but in the case of the Quebec bridge, where the unit strains are unusually high, approaching the elastic limit, the net areas of the members should be used in estimating the safe limit. Some of the compression members consist of sections which are composed of angles and a number of plates riveted together. The rivet holes reduce the sectional area, and, while these holes are filled up with rivets, they do not fill the holes so perfectly as to make them take the place of the material punched out of the rivet holes. In some of the lower chord members, the net section is about 86 per cent of the gross section, and the elastic limit, which is estimated to be 32,000 pounds per square inch, is thereby reduced to about 27,500 pounds per square inch of gross area. If we, therefore, assume the maximum permissible unit strain on the gross section for the specified extreme loading as 24,000 pounds per square inch, and the secondary strains as only 3,000 pounds per square inch, or approximately 12½ per cent of the direct strain, the total fibre strain per square inch would be $24{,}000 + 3{,}000 = 27{,}000$ pounds. This strain nearly reaches the elastic limit of 27,500 pounds per square inch with scarcely any margin for safety.

The maximum permissible strain of 24,000 pounds per square inch for the direct compression caused by the extreme load would have to be reduced in accordance with the accepted formulæ for compression members, making it $24{,}000 - 100\ {}^{l}/_{r}$; where l = length, and r = least radius of gyration of member.

For the working load there should be the same margin for safety as in tension members. As stated before, the elastic limit in compression members, owing to the reduction of their sections by the rivet holes, may be reduced to 27,500 pounds per square inch of gross section. Deducting 3,000 pounds per square inch for secondary strains would leave 24,500 pounds per square inch on the gross section as the maximum strain in direct compression within the elastic limit. Allowing 6/7 of this strain the same as for tension members, we have 21,000 pounds per square inch as permissible strain for direct compression, which should be reduced by the usual formulæ, making it $21{,}000 - 90\ {}^{l}/_{r}$. These limiting strains should be applied to all compression members. The writer does not advocate these high unit strains, but only desires to fix a limit within which the strains may be considered safe, and which could be used in comparison with the tables in Appendix B.

The extreme unit strains within which in the writer's judgment the structure may be considered to be able to sustain the loads provided for in the specifications are:—

First. For the dead and live loads combined with the snow load: For tension, 21,000 pounds per square inch of net section; for compression, $21{,}000 - 90\ {}^{l}/_{r}$ per square inch of gross section.

Second. For the extreme provision of 1½ times the live load, dead and snow loads, combined with ½ of the wind strains: For tension, 24,000 pounds per square inch of net section; for compression, $24{,}000 - 100\ {}^{l}/_{r}$ per square inch of gross section.

The table included in Appendix B gives these unit strains for different ratios of ${}^{l}/_{r}$.

By applying the above unit strains to the trusses of the cantilever and anchor arms in the present design of the Quebec bridge, we find the following discrepancies:

CANTILEVER AND ANCHOR ARMS.

Upper Chords.

The upper chords are composed of eyebars for which the maximum permissible strain as stated above should not exceed 21,000 pounds per square inch for the working load, and 24,000 pounds per square inch for the extreme load.

The tables in Appendix B show that the strains in all panels, excepting those from U_2 to U_6 of the cantilever arm, are in excess of these limits for either case of loading.

Lower Chords.

The lower chord in itself is not pin-connected, but is composed of a number of sections butting against each other and connected with splice plates. If the lower chords of the cantilever and anchor arms were strictly pin-connected, that is, bearing against the pin only, the strains would act in the axis of the member without any other bending movements from the dead load than those caused by the friction of the pin in the pin hole, as they would be able to rotate around the pins and thus adjust themselves during erection.

If the lower chords were continuous members and fully spliced, and the web members rigidly connected to them similar to those of the Firth of Forth bridge or the suspended span, the strains produced by the deformation would become an important factor, but could be approximately calculated and provided for in the sections. Since, however, the lower chord members of the Quebec bridge are butt-jointed, they are neither continuous nor pin-connected, and it is impossible to make the whole section bear uniformly under the various conditions of loading.

With accurate workmanship and proper method of erection, the joints of the chord members may come to a full and even bearing for one condition of loading, and in this condition the strains would be transmitted from one section to another in the direction of their axis and distributed over their entire cross-section. For all other conditions of loading, the strains are transmitted eccentrically, thus producing secondary strains in addition to the direct strains and those produced by the initial eccentricity inherent in all compression members. These secondary strains will be found in Appendix D accompanying this report.

By comparing the strains in the tables in Appendix B with the limits fixed by the writer, we find that all the lower chord members are deficient (with the exception of L_0 to L_4 of the cantilever arm) and would not be strong enough to safely carry the specified loads provided for in the specifications, even if they had been properly braced with lattice bars of sufficient strength; and that the inadequate latticing shown on the drawings would still further reduce their strength.

Web System:—

The web system of the trusses of the cantilever and anchor arms is composed of tension and compression members. The main posts are pin-connected to the upper and lower chords, while the web members among themselves are only partly pin-connected; that is, the diagonals, with the exception of the one nearest the center post, are eyebars and pin-connected at both ends.

Some of the sub-diagonals and floor-beam suspenders are compression and others tension members. The connections of the sub-diagonals are riveted at both ends. The floor-beam suspenders are pin-connected to the lower chord, but have riveted connections at their intersections with the main diagonals and sub-diagonals.

From the tables in Appendix B it is evident that the strains in the posts of the cantilever and anchor arms are excessive (with the exception of L_8-U_8), also in about one-half of the diagonals. The strains in the center posts are also excessive. The strains in the floor-beam suspenders, and in the sub-diagonals come practically within the safe limits.

Suspended Span:—

The trusses of the suspended span are practically riveted structures with the chords fully spliced and the web members rigidly connected, excepting the main or tension diagonals, which consist of eyebars pin-connected at their ends.

The weakest parts of the suspended span are the upper chords (see Appendix B), their unit strains being from 44 to 48 per cent in excess of the safe limits fixed by the writer. The strains in the lower chords and web members, excepting U_0-C_1 and C_1-L_2, are practically within those limits.

Sufficiency of Specifications:—

In considering the sufficiency of the specifications, the question arises: Would the trusses of the Quebec bridge have been safe if they had been designed to comply with the requirements of the specifications and the details had been in proportion to the strength of the members?

By referring to the tables in Appendix B, we find that the permissible unit strains limited by the specifications for the two kinds of loading, that is, the working load and the extreme load, are close to, or within the limits of those determined by the writer in all members of the trusses of the cantilever and anchor arms, except in the lower chords and in the posts over piers, for which strains are permitted beyond these limits.

In connection with this subject, the writer believes it to be within the scope of his investigations to report upon the specifications for the Quebec bridge.

The purpose of these specifications has evidently been to keep all the strains, even for the extreme loading, well within the elastic limit of the material. That this has not been realized in all the members of the structure is evident from a study of the tables in Appendix B. The writer has already given his reasons for recommending limiting unit strains, and has shown that the specifications permit too high unit strains for the posts over the piers and for the lower chord of the cantilever and anchor arms. The writer also considers the use of a formula for the permissible strains based on the minimum and maximum strains in each member, as given in the specifications of the Quebec bridge, to be unsuitable for practical purposes, as it is not supported by facts established by recent experiments, and causes unnecessary complications in the computation of the strength of the members; giving besides anomalous results.

The well-established theory of the elastic line is based on strains within the elastic limit. As a single strain above the elastic limit produces a permanent set and destroys the property of uniform elongation in the metal, its effect is not different from the effect of repeated strains, the single strain having practically destroyed the usefulness of the material. The elastic limit, therefore, is actually the ultimate strength for all practical purposes.

The static effect of a live load is the same as that of a dead load, depending upon the amount and distribution of the load only. The dynamic effect of a live load, commonly called impact, however, depends upon the conditions under which the live load is applied. The conditions which affect the impact on a railway bridge are the conditions of the track, the dynamic action produced by the deflection of the bridge, the action of insufficiently balanced drivers, the reciprocal motion and vibration of the machinery and the velocity of the train.

As the static and dynamic effects of a live load depend each upon such entirely different conditions, it seems rational to consider each separately in order to arrive at a more scientific solution of the problem of determining the safe working strains in railway bridges. As the internal strain of a member in a structure is proportional to its elongation or reduction in length, it is evident that it makes no difference, as far as the resistance of the material is concerned, whether this strain is produced by the weight of the structure, by the static effect of a superimposed load, or by the

dynamic effect of a moving load. If, therefore, the impact is added to the live load, reducing its effect to that of a static load, a uniform permissible strain may be used, thus avoiding complications and making the strength of the details and connections in proportion to that of the main members, as the impact applies to all parts.

RECOMMENDATIONS AS TO THE RECONSTRUCTION OF THE QUEBEC BRIDGE.

As it is evident from the writer's investigations that the trusses of the present design are not of sufficient strength to carry the loads provided for in the specifications, the question arises:

Can the fabricated members of the remaining half of the Quebec bridge, or a portion thereof, be utilized in the reconstruction of the bridge?

This might be accomplished in two different ways:

First.—By using the remaining portion of the floor system and reinforcing the remaining members of the trusses; rebuilding only that portion which has been wrecked.

The members composing the floor system and the lateral bracing of the remaining half of the bridge might be utilized in the reconstructed bridge. However, to make the bridge strong enough to carry the specified loads with a reasonable margin of safety, the sections of most of the members of the trusses would have to be increased. An examination of the detail plans of the members of the trusses from the standpoint of a manufacturer of structural steel work has convinced the writer that this is impracticable.

The weakest parts of the trusses of the anchor and cantilever arms are the lower chord members. Their sectional areas would have to be increased at least 50 per cent in order to reduce the unit strains to safe limits. The only way this could be done would be to cut them apart, drill additional rivet holes and rivet them up again with additional material. During these various manipulations the members would become distorted, and would require the reboring of the pin holes to larger size, and the refacing of the ends. This refacing would shorten the members enough to make them useless. The use of the remaining chord members is, therefore, impracticable. The same applies to most of the other compression members.

The upper chords of the cantilever and anchor arms being composed entirely of eyebars could be reinforced with additional bars, which would require in some panels as much as 20 per cent additional material. This operation would not only require new pins, but also the changing of the upper ends of the posts to which they are attached. The writer, therefore, considers it impracticable to use any of the finished truss members of the remaining half of the bridge.

Second.—By using the present floor system and building new trusses, following the same outlines as in the present design, but proportioning the members and connections for the loads provided for in the specifications.

If the remaining portion of the floor system and bracing, weighing about 8,000,000 pounds, were to be used in the new structure, it would require for the trusses a design similar to the present one, and also, the same distance between the posts to which the floorbeams are attached. This is almost an impossible task, and further as, in the writer's opinion, the present design of the trusses can be improved upon, the new design should be worked out on entirely different lines to avoid many of the complications and objectionable features existing in the present design.

A third proposition is to adopt an entirely new design, retaining only the length of span in order to use the present main piers, with some modifications. The anchorage piers would have to be partially rebuilt as new anchorages would be required.

Referring to the features which appear to be objectionable in the present design, the writer begs to call your attention to the following:—

The polygonal lower chords of the cantilever and anchor arms are not well adapted for a cantilever bridge on account of the difficulties in fabrication and proper fitting,

which make them not only more costly than chords forming a straight line, but also less safe. The polygonal chords of the present design produce a reversal of strains in some web members, which on that account require not only more material than members with strains in one direction only, but also cause unnecessary complications in their details and connections.

The wind forces in a rationally designed bridge should produce strains in the chords, the lateral and sway bracing only. On account of the shape of the chords of the Quebec Bridge, the wind strains affect also the web members of the trusses, producing in these members additional strains, consequently requiring in these members more material and more complicated details.

The writer considers that in a rationally designed structure the strains should be carried in the most direct line to the piers. The more complicated the design and the oftener the strains have to change their direction before reaching their destination, the more assumptions have to be made, which again reduce the degree of accuracy of the results of the computations; therefore, the simpler the design, the safer it will be with the same unit strains.

CONCLUSIONS.

The results of the writer's investigations and his recommendations may be briefly summarized as follows:—

First.—The floor system and bracing are of sufficient strength to safely carry the traffic for which they were intended.

Second.—The trusses, as shown in the design submitted to the writer, do not conform to the requirements of the approved specifications, and are inadequate to carry the traffic or loads specified.

Third.—The latticing of many of the compression members is not in proportion to the sections of the members which they connect.

Fourth.—The trusses of the bridge, even if they had been designed in accordance with the approved specifications, would not be of sufficient strength in all their parts to safely sustain the loads provided for in the specifications.

Fifth.—It is impracticable to use the fabricated material now on hand in the reconstruction of the bridge.

Sixth.—The present design is not well adapted to a structure of the magnitude of the Quebec Bridge and should, therefore, be discarded and a different design adopted for the new bridge, retaining only the length of the spans in order to use the present piers.

Seventh.—The writer considers the present piers strong enough to carry a heavier structure, assuming that the bearing capacity of the foundations is sufficient to sustain the increased pressure.

This report is accompanied by the following Appendices:

A.—Copy of revised specifications.

B.—Tables containing computations of strains in the members of the trusses, a table giving permissible strains for compression members, also diagrams of dead load concentrations and loads and strains during erection, August 29, 1907 (20 prints.)

C.—Review of the literature on the theory of compression members up to the present time.

D.—Investigation of secondary strains in trusses.

Respectfully submitted,

C. C. SCHNEIDER.

M. J. Butler, Esq.,
Deputy Minister and Chief Engineer,
Department of Railways and Canals.

APPENDIX A.

QUEBEC BRIDGE SPECIFICATIONS FOR LOADS AND STRAINS FOR CANTILEVER AND SUSPENDED SPANS, BY THEODORE COOPER.

FLOOR SYSTEM.

Railroad Stringers.—To be designed to carry Cooper's E-40 engines with unit strains not exceeding 10,000 lbs. per square inch of net section.

Trolley Stringers.—Loaded with cars weighing 56,000 lbs. on two axles ten feet apart, not to be strained above 13,000 lbs. per square inch of net section. Cars thirty feet over all.

Highway Stringers.—Loaded with 24,000 lbs. on two axles ten feet apart, strains not to exceed 15,000 lbs. per square inch of net section.

Transverse Floor Beams.—With all tracks loaded as above they must not be strained above 15,000 lbs. per square inch of net section, or 12,000 lbs. with both railroad tracks loaded.

The webs of all girders shall be considered as resisting shearing strains only and will not be estimated as doing any flange duty.

TRUSSES.

The maximum strains produced by the following live loads and wind shall be used for proportioning all members of the trusses or towers:—

1st. A continuous train of any length weighing 3,000 lbs. per foot of track, moving in either direction on each track.

2nd. A train nine hundred feet long consisting of two E-33 engines followed by a load of 3,300 lbs. per lin, ft. upon each railroad track and moving in either direction.

3rd. A train load 550 feet long consisting of one E-40 engine followed by 4,000 lbs. per lin. ft. of track, on each track.

4th. For the suspended span a lateral wind force of 700 lbs. per lin. ft. of the top chord and 1,700 lbs. per lin. ft. of the lower chord, one half of which shall be used for lateral and diagonal bracing.

For the cantilever and anchor arms a lateral force of 500 lbs. on the top chord and 1,000 lbs. on the lower chord, per lin. ft. in addition to the wind force on the suspended span, shall be considered.

Only one-third of this maximum wind force need be considered in proportioning the chords. It shall be considered as a live load. Unless this increases the strains due to the live and dead loads only more than 25 per cent the sections need not be increased.

Reversal of strains by the wind acting in opposite directions need not be considered; but where the maximum wind forces reverse the strains in any member the member must be designed to resist each kind of strain.

Allowed Working Strains.—Under the above working loads in combination with the dead loads, the allowed strains in all members of the trusses and towers shall not exceed the following limits:—

Tension Chords and Diagonals.—

$$12{,}000\left(1+\frac{\text{Min}}{\text{Max}}\right) \text{ lbs. per sq. in. of net section.}$$

Compression Chords.—(Where l does not exceed 50 r).

$$12{,}000\left(1+\frac{\text{Min}}{\text{Max}}\right) \text{ per sq. in.}$$

Main Posts.—

$$\left(12{,}000-50\frac{l}{r}\right)\left(1+\frac{\text{Min}}{\text{Max}}\right) \text{ lbs. per sq. in.}$$

TRUSSED FLOORBEAMS.

Tension Struts.—

$$10{,}000\left(1+\frac{\text{Min}}{\text{Max}}\right) \text{ for R. R. loading.}$$

$$12{,}000\left(1+\frac{\text{Min}}{\text{Max}}\right) \text{ for total loading.}$$

Compression Struts.—

$$\left(10{,}000-40\frac{l}{r}\right)\left(1+\frac{\text{Min}}{\text{Max}}\right) \text{ for R. R. loading.}$$

$$\left(12{,}000-50\frac{l}{r}\right)\left(1+\frac{\text{Max}}{\text{Min}}\right) \text{ for total loading.}$$

WIND STRUTS AND LATERALS.

Tension.—20,000 lbs. per sq. in.

Compression.—$20{,}000-90\dfrac{l}{r}$ per sq. in.

For counters and intermediate posts, the live load on the railroad tracks shall be increased 15 per cent.

COMBINED AND REVERSED STRAINS.

The allowed positive and negative strains upon any member subject to any combination of $\pm D$, $\pm L$, $\mp L'$ shall be determined by the following formulae:—

$$\text{Allowed} \pm \text{Strain, } 12{,}000\left(1+\frac{D-L}{D+L+L'}\right)$$

$$\text{Allowed} \mp \text{Strain, } 12{,}000\left(\frac{L'}{D+L+L'}\right)$$

PROVISION FOR FUTURE INCREASE OF LIVE LOAD.

In addition to the previous provisions as to the working loads and strains, no member of the trusses or towers shall be strained to exceed three-quarters of the elastic limit under the extreme assumption of an increase in the train loads of 50 per cent above those previously specified. Or, not to exceed 24,000 for the chords and main diagonals, or $24{,}000-100\dfrac{l}{r}$ for the posts.

The material to be medium steel of the best quality and made by the open hearth process.

All details, proportion of parts, workmanship, &c., to be in accordance with the best accepted practice.

Corrected to date, March 2, 1904.

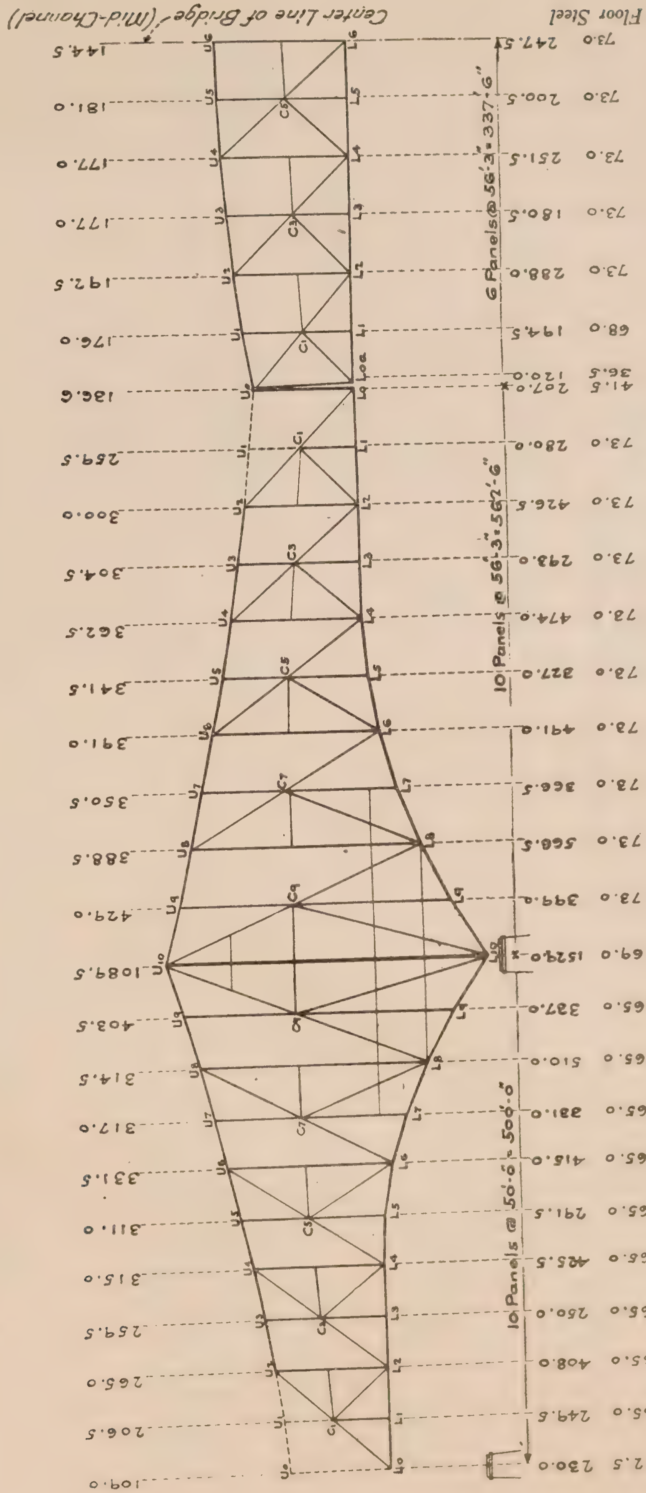
Appenda. June 13, 1905.

For the cantilever arms, the full wind on the suspended span should be considered. A snow load of 1,600 lbs. per foot of bridge should be used.

Appendix B.

Report on Quebec Bridge.

Dead Load Concentrations at Panel Points.



Weights are for one Truss and are given in units of 1000#

APPENDIX B.

REPORT ON QUEBEC BRIDGE.

TABULATED STATEMENT OF STRAINS, SECTIONAL AREAS AND UNIT STRAINS.

The strains in the members are given in thousands of pounds, the unit strains in pounds, and the following notations are used;-

+ denotes Tension,
— " Compression,
A " Sectional Area of Member, in sq. inches,
r " Least Radius of Gyration of Member, in inches,
D " Strain resulting from Dead Load,
L " " " " Live ",
S " " " " Snow ",
W " " " " Wind Pressure,
E " Maximum Strain occurring August 29, 1907.
ue " Unit Strain " " "

* φ denotes coefficient by which the specified minimum unit strain of 12000 lbs. pr. sq. in. for Tension, or $(12000 - 50\frac{l}{r})$ lbs. pr. sq. in for Compression, is to be multiplied in order to ascertain the permissible unit strain.

u denotes unit strain for Dead, Live and Snow Loads,
I- as required by Specifications;
II- as would actually occur in completed structure.

u₁ denotes unit strain for dead, 1½ live and snow loads, combined with ⅓ wind pressure,
I- as required by Specifications,
II- as would actually occur in completed structure.

* For strains of one kind only, $\varphi = 1 + \frac{min.}{max.}$
For combined Strains, $\varphi = 1 + \frac{D - L_1}{D + L + L_1}$
For reversed Strains, $\varphi = \frac{L_1}{D + L + L_1}$

Report on Quebec Bridge. Appendix B.

Anchor Arm - Upper Chord.

Member-	U_2-U_3	U_3-U_4	U_4-U_5	U_5-U_6	U_6-U_7	U_7-U_8	U_8-U_9	U_9-U_{10}
A	309	309	555	559	696	698	707	711
D	+ 4290	+ 4305	+ 8500	+ 8535	+ 11510	+ 11555	+ 12510	+ 12585
L	+ 1998	+ 2006	+ 3475	+ 3490	+ 4156	+ 4176	+ 4047	+ 4068
	− 749	− 751	− 968	− 973	− 772	− 776	− 380	− 382
S	+ 365	+ 365	+ 700	+ 705	+ 915	+ 915	+ 960	+ 965
W	+ 70	+ 10	+ 130	0	+ 150	+ 320	+ 1200	+ 1400
D+L+S	+ 6653	+ 6676	+ 12675	+ 12730	+ 16581	+ 16646	+ 17517	+ 17618
$D+1\frac{1}{2}L+S+\frac{1}{3}W$	+ 7675	+ 7682	+ 14455	+ 14475	+ 18709	+ 18841	+ 19940	+ 20119
φ	$1+\frac{3541}{7037}=1.50$	$1+\frac{3554}{7062}=1.50$	$1+\frac{7532}{12943}=1.58$	$1+\frac{7562}{12998}=1.58$	$1+\frac{10738}{16438}=1.65$	$1+\frac{10779}{16507}=1.65$	$1+\frac{12130}{16937}=1.72$	$1+\frac{12203}{17035}=1.72$
u I	18000	18000	19000	19000	19800	19800	20600	20600
u II	21500	21600	22800	22800	23800	23800	24800	24800
u_1 I	24000	24000	24000	24000	24000	24000	24000	24000
u_1 II	24800	24900	26100	25900	26900	27000	28200	28300
E	+ 4185	+ 4205	+ 8245	+ 8280	+ 11095	+ 11150	+ 12020	+ 12090
u_e	13500	13600	14900	14800	15900	16000	17000	17000

REPORT ON QUEBEC BRIDGE. APPENDIX B.

ANCHOR ARM - LOWER CHORD.

Member-		Lo-L1	L1-L2	L2-L3	L3-L4	L4-L5	L5-L6	L6-L7	L7-L8	L8-L9	L9-L10
A		302	302	542	542	702	709	729	768	781	843
$\frac{l}{r}$		$\frac{600}{18.7}=32.1$	$\frac{600}{18.7}=32.1$	$\frac{600}{16.5}=36.3$	$\frac{600}{16.5}=36.3$	$\frac{600}{16.2}=37.0$	$\frac{609}{16.2}=37.6$	$\frac{626}{16.2}=38.6$	$\frac{652}{16.2}=40.2$	$\frac{684}{16.2}=42.2$	$\frac{722}{16.2}=44.5$
D		− 3985	− 3985	− 8110	− 8110	− 11425	− 11585	− 12755	− 13275	− 13690	− 14455
L		− 1965	− 1965	− 3413	− 3413	− 4212	− 4270	− 4181	− 4349	− 4021	− 4249
		+ 840	+ 840	+ 1037	+ 1037	+ 840	+ 852	+ 438	+ 455	+ 43	+ 45
S		− 335	− 335	− 670	− 670	− 910	− 920	− 985	− 1025	− 1025	− 1080
W		− 660	− 1420	− 2430	− 3260	− 4170	− 5160	− 5570	− 6870	− 7370	− 9060
D+L+S		− 6285	− 6285	− 12193	− 12193	− 16547	− 16775	− 17921	− 18649	− 18736	− 19784
$D+1\frac{1}{2}L+S+\frac{1}{3}W$		− 7487	− 7740	− 14709	− 14986	− 20043	− 20630	− 21868	− 23113	− 23203	− 24928
φ		$1+\frac{3145}{6790}=1.46$	$1+\frac{3145}{6790}=1.46$	$1+\frac{7073}{12560}=1.56$	$1+\frac{7073}{12560}=1.56$	$1+\frac{10585}{16477}=1.64$	$1+\frac{10733}{16707}=1.64$	$1+\frac{12317}{17374}=1.71$	$1+\frac{12820}{18079}=1.71$	$1+\frac{13647}{17754}=1.77$	$1+\frac{14410}{18749}=1.77$
u	I	17500	17500	18700	18700	19700	19700	20500	20500	21200	21200
	II	20800	20800	22500	22500	23600	23700	24600	24300	24000	23500
u_1	I	24000	24000	24000	24000	24000	24000	24000	24000	24000	24000
	II	24800	25600	27100	27600	28600	29100	30000	30100	29700	29600
E		− 3915	− 3915	− 7885	− 7885	− 11050	− 11200	− 12260	− 12755	− 13125	− 13870
u_e		13000	13000	14500	14500	15700	15800	16800	16600	16800	16500

REPORT ON QUEBEC BRIDGE. APPENDIX B.

Anchor Arm – Vertical Posts.

Member		L_2–U_2	L_4–U_4	L_6–U_6	L_8–U_8 Upper	L_8–U_8 Middle	L_8–U_8 Lower	L_{10}–U_{10} Upper	L_{10}–U_{10} Middle	L_{10}–U_{10} Below Floor.	L_{10}–U_{10} Lower
A		371	355	277	175	163	163	472	514	514	472
$\frac{l}{r}$		$\frac{720}{14.5} = 50$	$\frac{837}{14.6} = 57$	$\frac{1058}{15.3} = 69$	$\frac{1169}{16.15} = 72$	$\frac{920}{16.3} = 56$	$\frac{598}{16.3} = 37$	$\frac{759}{17.5} = 43$	$\frac{920}{18} = 51$	$\frac{596}{18} = 33$	$\frac{742}{17.5} = 42$
D		− 4180	− 4480	− 3510	− 1225	− 1225	− 1380	− 8065	− 8065	− 8250	− 8250
L		− 1848	− 1499	− 895	− 633	− 633	− 827	− 1444	− 1444	− 1619	− 1619
		+ 692	+ 310	+ 67	+ 323	+ 323	+ 323				
S		− 335	− 325	− 220	− 25	− 25	− 65	− 375	− 375	− 420	− 420
W		− 270	− 230	− 200	− 810	− 810	− 810	− 4830	− 4830	− 4830	− 4830
D+L+S		− 6363	− 6304	− 4625	− 1883	− 1883	− 2272	− 9884	− 9884	− 10289	− 10289
$D+1\frac{1}{2}L+S+\frac{1}{3}W$		− 7377	− 7130	− 5139	− 2469	− 2469	− 2955	− 12216	− 12216	− 12708	− 12708
φ		$1+\frac{3488}{6720}=1.52$	$1+\frac{4170}{6289}=1.66$	$1+\frac{3443}{4472}=1.77$	$1+\frac{902}{2181}=1.41$	$1+\frac{902}{2181}=1.41$	$1+\frac{1057}{2530}=1.42$	$1+\frac{8065}{9509}=1.85$	$1+\frac{8065}{9509}=1.85$	$1+\frac{8250}{9869}=1.836$	$1+\frac{8250}{9869}=1.836$
u	I	14400	15200	15100	11900	13000	14400	18200	17500	18900	18200
	II	17200	17800	16700	10800	11600	13900	20900	19200	20000	21800
u_1	I	19000	18300	17100	16800	18400	20300	19700	18900	20700	19800
	II	19900	20100	18500	14100	15100	18100	25900	23800	24700	26900
E		− 4155	− 4475	− 3550	− 1365	− 1365	− 1470	− 7380	− 7380	− 7510	− 7510
u_e		11200	12600	12800	7800	8400	9000	15600	14400	14600	15900

REPORT ON QUEBEC BRIDGE. APPENDIX B.

Anchor Arm – Main Diagonals.

Member-	L_0-C_1	C_1-U_2	L_2-C_3	C_3-U_4	L_4-C_5	C_5-U_6	L_6-C_7	C_7-U_8	L_8-C_9	C_9-U_{10}
A	451	454	396	400	330	300	150	149	163	225
$\frac{l}{r}$									$\frac{984}{14.8}=66$	$\frac{816}{13.8}=59$
D	+ 5975	+ 6325	+ 6335	+ 6650	+ 5595	+ 5115	+ 2285	+ 1815	+ 35	− 360
L	+ 2944	+ 2944	+ 2357	+ 2332	+ 1550	+ 1371	+ 1731	+ 832	+ 914	+ 994
	− 1258	− 1105	− 616	− 483	− 187	− 107	− 62	− 326	− 961	− 1179
S	+ 515	+ 540	+ 505	+ 530	+ 410	+ 365	+ 125	+ 80	− 70	− 105
W	+ 420	+ 410	+ 340	+ 330	+ 40	+ 280	+ 770	+ 1005	− 990	− 2350
D+L+S	+ 9434	+ 9809	+ 9197	+ 9512	+ 7555	+ 6851	+ 3141	+ 2727	− 996	− 1644
$D+1\frac{1}{2}L+S+\frac{1}{3}W$	+ 11046	+ 11418	+ 10488	+ 10788	+ 8343	+ 7629	+ 3763	+ 3478	− 2139	− 3016
φ	$1+\frac{4717}{10177}=1.464$	$1+\frac{5220}{10374}=1.503$	$1+\frac{5719}{9308}=1.614$	$1+\frac{6167}{9465}=1.651$	$1+\frac{5408}{7332}=1.738$	$1+\frac{5008}{6593}=1.760$	$1+\frac{2223}{3078}=1.722$	$1+\frac{1489}{2973}=1.500$	$\frac{961}{1910}=0.503$	$\frac{994}{2533}=0.392$
u I	17600	18000	19400	19800	20800	21100	20700	18000	4380	3550
u II	20900	21600	23200	23800	22900	22800	20900	18300	6110	7300
u_1 I	24000	24000	24000	24000	24000	24000	24000	24000	17400	18100
u_1 II	24500	25100	26500	27000	25300	25400	25100	23300	13100	13400
E	+ 5885	+ 6200	+ 6145	+ 6435	+ 5380	+ 4870	+ 2150	+ 1675	− 25	− 415
u_e	13000	13700	15500	16100	16300	16200	14300	11200	150	1800

REPORT ON QUEBEC BRIDGE.

APPENDIX B.

Anchor Arm – Suspenders.

Member	L_1-C_1	L_3-C_3	L_5-C_5	L_7-C_7 Upper	L_7-C_7 Lower	L_9-C_9 Upper	L_9-C_9 Middle	L_9-C_9 Lower
A	41 net	41 net	149 gr.	141 gr.	97 gr.	125 gr.	108 gr.	.96 gr.
$\frac{l}{r}$			$\frac{933}{10.7}=87$	$\frac{974}{10.8}=90$	$\frac{333}{11.4}=29$	$\frac{927}{11.0}=84$	$\frac{598}{11.4}=52$	$\frac{328}{11.7}=28$
D	+ 315	+ 315	− 1055	− 1115	− 1270	− 1080	− 1245	− 1245
L	+ 412	+ 412	− 521	− 494	− 494	− 436	− 436	− 436
			+ 497	+ 458	+ 70	+ 420	+ 8	+ 8
S	+ 40	+ 40	− 70	− 75	− 115	− 70	− 110	− 110
W	0	0	− 570	− 720	− 720	− 870	− 870	− 870
D+L+S	+ 767	+ 767	− 1646	− 1684	− 1879	− 1586	− 1791	− 1791
$D+1\frac{1}{2}L+S+\frac{1}{3}W$	+ 973	+ 973	− 2096	− 2171	− 2366	− 2094	− 2299	− 2299
φ	$1+\frac{315}{727}=1.433$	$1+\frac{315}{727}=1.433$	$1+\frac{558}{2073}=1.269$	$1+\frac{657}{2067}=1.318$	$1+\frac{1200}{1834}=1.654$	$1+\frac{660}{1936}=1.341$	$1+\frac{1237}{1689}=1.732$	$1+\frac{1237}{1689}=1.732$
u I	17200	17200	9700	9900	17500	10500	16300	18400
u II	18700	18700	11000	11900	19400	12700	16600	18700
u_1 I	24000	24000	15300	15000	21100	15600	18800	21200
u_1 II	23700	23700	14100	15400	24400	16700	21300	24000
E	+ 260	+ 260	− 1060	− 1110	− 1215	− 1085	− 1200	− 1200
u_e	6300	6300	7100	7900	12500	8700	11100	12500

REPORT ON QUEBEC BRIDGE. APPENDIX B.

Anchor Arm - Diagonal Sub struts.

Member		C1-L2	C3-L4	C5-L6	C7-L8	C9-L10
A		51.4 gr.	51.4 gr.	59.6 net	59.6 net	52.6 net
$\frac{l}{r}$		$\frac{889}{11.3} = 79$	$\frac{969}{11.3} = 86$			
D		− 345	− 310	+ 535	+ 620	+ 555
L		− 276	− 255	+ 363	+ 355	+ 317
		−	+ 25	− 319	− 275	− 250
S		− 25	− 25	+ 50	+ 55	+ 55
W		0	0	+ 340	+ 430	+ 500
D+L+S		− 646	− 590	+ 948	+ 1030	+ 927
$D+1\frac{1}{2}L+S+\frac{1}{3}W$		− 784	− 717	+ 1242	+ 1350	+ 1252
φ		$1+\frac{345}{621}=1.556$	$1+\frac{285}{590}=1.483$	$1+\frac{216}{1217}=1.177$	$1+\frac{345}{1250}=1.276$	$1+\frac{305}{1122}=1.272$
u	I	12500	11400	14100	15300	15300
	III	12600	11500	15900	17300	17600
u_1	I	16100	15400	24000	24000	24000
	III	15200	13900	20800	22600	23800
E		− 315	− 280	+ 560	+ 620	+ 565
u_e		6100	5400	9400	10400	10700

REPORT ON QUEBEC BRIDGE. APPENDIX B

Cantilever Arm - Upper Chord.

Member.	U_2-U_3	U_3-U_4	U_4-U_5	U_5-U_6	U_6-U_7	U_7-U_3	U_8-U_9	U_9-U_{10}
A	435	437	572	574	649	650	664	669
D	+ 3540	+ 3550	+ 7420	+ 7440	+ 10530	+ 10560	+ 11900	+ 11950
L	+ 1505	+ 1508	+ 2773	+ 2784	+ 3564	+ 3576	+ 3745	+ 3760
S	+ 325	+ 325	+ 640	+ 645	+ 860	+ 865	+ 930	+ 930
W	+ 350	+ 352	+ 608	+ 610	+ 728	+ 731	+ 990	+ 1140
D+L+S	+ 5370	+ 5383	+ 10833	+ 10869	+ 14954	+ 15001	+ 16575	+ 16640
$D+1\frac{1}{2}L+S+\frac{1}{3}W$	+ 6239	+ 6254	+ 12422	+ 12464	+ 16979	+ 17033	+ 18777	+ 18900
φ	$1+\frac{3540}{5045}=1.702$	$1+\frac{3550}{5058}=1.702$	$1+\frac{7420}{10193}=1.728$	$1+\frac{7440}{10224}=1.728$	$1+\frac{10530}{14094}=1.747$	$1+\frac{10560}{14136}=1.747$	$1+\frac{11900}{15645}=1.760$	$1+\frac{11950}{15710}=1.760$
u { I	20400	20400	20700	20700	21000	21000	21100	21100
u { II	12300	12300	18900	18900	23000	23100	25000	24900
u_1 { I	24000	24000	24000	24000	24000	24000	24000	24000
u_1 { II	14300	14300	21700	21700	26200	26200	28300	28300
E	+ 4405	+ 4410	+ 7890	+ 7910	+ 10575	+ 10610	+ 11635	+ 11680
u_e	10100	10100	13800	13800	16300	16300	17500	17500

REPORT ON QUEBEC BRIDGE. APPENDIX B

Cantilever Arm - Lower Chord

Member	L_0-L_1	L_1-L_2	L_2-L_3	L_3-L_4	L_4-L_5	L_5-L_6	L_6-L_7	L_7-L_8	L_8-L_9	L_9-L_{10}
A	456	456	602	602	708	728	728	767	767	841
$\frac{l}{r}$	$\frac{675}{17.7} = 38$	$\frac{675}{17.7} = 38$	$\frac{675}{16.5} = 41$	$\frac{675}{16.5} = 41$	$\frac{676}{16.2} = 42$	$\frac{685}{16.2} = 42$	$\frac{702}{16.2} = 43$	$\frac{726}{16.1} = 45$	$\frac{756}{16.1} = 47$	$\frac{792}{16.1} = 49$
D	− 3205	− 3205	− 7060	− 7060	− 10515	− 10655	− 12220	− 12630	− 13410	− 14050
L	− 1443	− 1443	− 2733	− 2733	− 3652	− 3699	− 3910	− 4042	− 3951	− 4140
S	− 295	− 295	− 610	− 610	− 865	− 875	− 960	− 990	− 1005	− 1055
W	− 700	− 1427	− 2453	− 3275	− 4154	− 5136	− 5518	− 6790	− 7244	− 8883
D+L+S	− 4943	− 4943	− 10403	− 10403	− 15032	− 15229	− 17090	− 17662	− 18366	− 19245
$D+1\frac{1}{4}L+S+\frac{1}{3}W$	− 5879	− 6140	− 12587	− 12861	− 18243	− 18790	− 20884	− 21946	− 22756	− 24276
φ	$1+\frac{3205}{4648}=1.690$	$1+\frac{3205}{4648}=1.690$	$1+\frac{7060}{9793}=1.721$	$1+\frac{7060}{9793}=1.721$	$1+\frac{10515}{14167}=1.742$	$1+\frac{10655}{14354}=1.742$	$1+\frac{12220}{16130}=1.757$	$1+\frac{12630}{16672}=1.757$	$1+\frac{13410}{17361}=1.772$	$1+\frac{14050}{18190}=1.772$
u I	20300	20300	20600	20600	20900	20900	21100	21100	21300	21300
u II	10800	10800	17300	17300	21200	20900	23500	23000	24000	22900
u_1 I	24000	24000	24000	24000	24000	24000	24000	24000	24000	24000
u_1 II	12900	13500	20900	21400	25800	25800	28700	28600	29700	28900
E	− 3860	− 3860	− 7545	− 7545	− 10605	− 10740	− 11965	− 12380	− 12925	− 13545
u_e	8500	8500	12500	12500	15000	14800	16400	16100	16900	16100

REPORT ON QUEBEC BRIDGE.

APPENDIX B

Cantilever Arm—Vertical Posts.

Member.		Lo-Uo	L2-U2	L4-U4	L6-U6	L8-U8 Upper	L8-U8 Middle	L8-U8 Lower.
A		249	241	283	259	196	184	184
$\frac{l}{r}$		$\frac{1086}{16.9} = 64$	$\frac{718}{15.6} = 46$	$\frac{835}{15.3} = 55$	$\frac{1060}{15.4} = 69$	$\frac{1174}{16.6} = 71$	$\frac{918}{16.8} = 55$	$\frac{601}{16.8} = 36$
D		− 2905	− 3210	− 3880	− 3480	− 1840	− 1840	− 2025
L		− 2235	− 1250	− 1285	− 962	− 570	− 570	− 854
						+ 133	+ 133	+ 133
S		− 265	− 265	− 285	− 215	− 45	− 45	− 90
W		− *570 − 340	− 280	− 240	− 190	− 990	− 990	− 990
D + L + S		− 5405	− 4725	− 5450	− 4657	− 2455	− 2455	− 2969
D + $1\frac{1}{2}$L + S + $\frac{1}{3}$W		− 6825	− 5443	− 6172	− 5201	− 3070	− 3070	− 3726
φ		$1+\frac{2905}{5140}=1.565$	$1+\frac{3210}{4460}=1.720$	$1+\frac{3880}{5165}=1.751$	$1+\frac{3480}{4442}=1.783$	$1+\frac{1707}{2543}=1.671$	$1+\frac{1707}{2543}=1.671$	$1+\frac{1892}{3012}=1.628$
u	I	13800	16700	16200	15300	14100	15500	16600
	II	21700	19600	19200	18000	12500	13300	16100
u_1	I	17600	19400	18500	17100	16900	18500	20400
	II	27400	22600	21800	20100	15700	16700	20200
E		− 1600	− 2220	− 3565	− 3140	− 1590	− 1590	− 1715
u_e		6400	9200	12600	12100	8100	8600	9300

* Bending.

REPORT ON QUEBEC BRIDGE. APPENDIX B.

Cantilever Arm - Main Diagonals.

Member-	Lo-C1	C1-U2	L2-C3	C3-U4	L4-C5	C5-U6	L6-C7	C7-U8	L8-C9	C9-U10
A	300	315	330	345	300	300	180	180	191 gr. 174 net	226 gr. 205 net
$\frac{l}{r}$									$\frac{984}{14.5} = 68$	$\frac{816}{13.6} = 60$
D	+ 4470	+ 4910	+ 5355	+ 5760	+ 5260	+ 4970	+ 2760	+ 2450	+ 950	+ 690
L	+ 2007	+ 2085	+ 2043	+ 2224	+ 1678	+ 1524	+ 805	+ 791	+ 673	+ 684
							− 26	− 202	+ 655	− 821
S	+ 420	+ 450	+ 445	+ 470	+ 380	+ 350	+ 150	+ 115	+ 15	+ 45
W	+ 487	+ 487	+ 391	+ 387	+ 19	+ 280	+ 1040	+ 1350	± 1840	± 2154
D+L+S	+ 6897	+ 7445	+ 7843	+ 8454	+ 7318	+ 6844	+ 3715	+ 3356	+ 1623	+ 1374
$D+1\frac{1}{2}L+S+\frac{1}{3}W$	+ 8062	+ 8649	+ 8994	+ 9695	+ 8163	+ 7699	+ 4464	+ 4201	+ 2572	+ 2434
φ	$1+\frac{4470}{6477}=1.691$	$1+\frac{4910}{6995}=1.702$	$1+\frac{5355}{7398}=1.724$	$1+\frac{5760}{7984}=1.721$	$1+\frac{5260}{6938}=1.758$	$1+\frac{4970}{6494}=1.765$	$1+\frac{2734}{3591}=1.761$	$1+\frac{2248}{3443}=1.653$	$1+\frac{295}{2278}=1.130$	$\frac{821}{2195} = 0.374$
u I	20300	20400	20700	20700	21100	21200	21100	19800	13600	4490
u II	23000	23600	23800	24500	24400	22800	20600	18600	9300	6700
u_1 I	24000	24000	24000	24000	24000	24000	24000	24000	24000	24000
u_1 II	26900	27400	27200	28100	27200	25700	24800	23300	14800	11900
E	+ 2560	+ 3260	+ 4795	+ 5155	+ 4630	+ 4280	+ 2210	+ 1860	+ 515	+ 235
u_e	8500	10300	14500	14900	15400	14300	12300	10300	3000	1100

Report on Quebec Bridge.

Appendix B.

Cantilever Arm - Suspenders.

Member-	L_1-C_1	L_3-C_3	L_5-C_5	L_7-C_7 Upper	L_7-C_7 Lower	L_9-C_9 Upper	L_9-C_9 Lower
A	60 net	60 net	117 gr.	117 gr.	89 gr.	117 gr.	93 gr.
$\frac{l}{r}$			$\frac{937}{11.2}=84$	$\frac{974}{11.2}=87$	$\frac{335}{11.7}=29$	$\frac{918}{11.2}=82$	$\frac{601}{11.6}=52$
D	+ 355	+ 355	− 735	− 850	− 1035	− 845	− 1040
L	+ 458	+ 458	− 402	− 414	− 414	− 388	− 388
			+ 412	+ 415			
S	+ 45	+ 45	− 55	− 60		+ 416	
W	0	0	− 507	− 650	− 100	− 60	− 100
D + L + S	+ 858	+ 858	− 1192	− 1324	− 650	− 771	− 771
$D+1\frac{1}{2}L+S+\frac{1}{3}W$	+ 1087	+ 1087	− 1562	− 1748	− 1549	− 1293	− 1528
					− 1973	− 1744	− 1979
φ	$1+\frac{355}{813}=1.437$	$1+\frac{355}{813}=1.437$	$1+\frac{323}{1549}=1.208$	$1+\frac{435}{1679}=1.259$	$1+\frac{1035}{1449}=1.714$	$1+\frac{429}{1649}=1.260$	$1+\frac{1040}{1428}=1.728$
u I	17200	17200	9400	10000	18100	10000	16200
u II	14300	14300	10200	11300	17400	11000	16400
u_1 I	24000	24000	15600	15300	21100	15800	18800
u_1 II	18100	18100	13300	14900	22200	14900	21300
E	+ 760	+ 300	− 825	− 900	− 1030	− 860	− 995
u_e	12700	5000	7100	7700	11600	7400	10700

REPORT ON QUEBEC BRIDGE.

APPENDIX B.

Cantilever Arm - Diagonal Substruts.

Member		C1–L2	C3–L4	C5–L6	C7–L8	C9–L10
A		64 gr.	64 gr.	64 net	64 net	64 net.
$\frac{l}{r}$		$\frac{951}{11.4} = 83$	$\frac{1029}{11.4} = 90$			
D		− 440	− 405	+ 325	+ 420	+ 380
L		− 328	− 306	+ 283	+ 300	+ 279
			+ 16	− 265	− 264	− 261
S		− 30	− 25	+ 40	+ 50	+ 45
W		0	− 4	+ 332	+ 412	+ 465
D+L+S		− 798	− 736	+ 648	+ 770	+ 704
$D+1\frac{1}{2}L+S+\frac{1}{3}W$		− 962	− 890	+ 900	+ 1057	+ 998
φ		$1+\frac{440}{768} = 1.573$	$1+\frac{389}{727} = 1.535$	$1+\frac{60}{873} = 1.069$	$1+\frac{156}{984} = 1.158$	$1+\frac{119}{920} = 1.129$
u	I	12300	11500	12800	13900	13500
	II	12500	11500	10100	12000	11000
u_1	I	15700	15000	24000	24000	24000
	II	15000	13900	14100	16500	15600
E		− 705	− 360	+ 390	+ 455	+ 395
u_e		11000	5600	6100	7100	6200

REPORT ON QUEBEC BRIDGE. APPENDIX B.

Suspended Span–Upper Chord

Member	$U_0 U_1$	$U_1 U_2$	$U_2 U_3$	$U_3 U_4$	$U_4 U_5$	$U_5 U_6$
A	158 gr. 136 net	158 gr. 136 net	224 gr. 191 net	224 gr. 191 net	242 gr. 207 net	242 gr. 207 net
$\frac{l}{r}$	$\frac{686}{13.9} = 49$	$\frac{683}{13.9} = 49$	$\frac{680}{13.5} = 50$	$\frac{677}{13.5} = 50$	$\frac{676}{13.5} = 50$	$\frac{675}{13.5} = 50$
D	− 2400	− 2385	− 3415	− 3400	− 3700	− 3695
L	− 1098	− 1091	− 1582	− 1577	− 1717	− 1714
S	− 225	− 225	− 325	− 325	− 350	− 350
W	− 181	− 330	− 445	− 528	− 577	− 594
D+L+S	− 3723	− 3701	− 5322	− 5302	− 5767	− 5759
$D+1\frac{1}{2}L+S+\frac{1}{3}W$	− 4332	− 4356	− 6261	− 6266	− 6817	− 6814
φ	$1+\frac{2400}{3498}=1.686$	$1+\frac{2385}{3476}=1.686$	$1+\frac{3415}{4997}=1.683$	$1+\frac{3400}{4977}=1.683$	$1+\frac{3700}{5417}=1.683$	$1+\frac{3695}{5409}=1.683$
u	20200	20200	20200	20200	20200	20200
	23600	23400	23800	23700	23800	23800
u_1	24000	24000	24000	24000	24000	24000
	27400	27600	28000	28000	28200	28200
E	+ 380	+ 380				
u_e	2800	2800				

Report on Quebec Bridge. Appendix B.

Suspended Span - Lower Chord.

Member		Loa-L1	L1-L2	L2-L3	L3-L4	L4-L5	L5-L6	
A		251 gr. 220 net	251 gr. 220 net	206 gr. 186 net	206 gr. 186 net	260 net	260 net	
$\frac{l}{r}$		$\frac{588}{12.15} = 48$	$\frac{675}{12.15} = 56$	$\frac{675}{12.6} = 54$	$\frac{675}{12.6} = 54$			
D		+ 210	+ 210	+ 2500	+ 2500	+ 3550	+ 3550	
L		+ 261	+ 261	+ 1167	+ 1167	+ 1646	+ 1646	
		− 18	− 18					
S		+ 20	+ 20	+ 235	+ 235	+ 335	+ 335	
W		+ 227	+ 625	+ 947	+ 1189	+ 1350	+ 1431	
D + L + S		+ 491	+ 491	+ 3902	+ 3902	+ 5531	+ 5531	
$D + 1\frac{1}{2}L + S + \frac{1}{3}W$		+ 697	+ 829	+ 4801	+ 4881	+ 6804	+ 6831	
φ		$1 + \frac{192}{489} = 1.393$	$1 + \frac{192}{489} = 1.393$	$1 + \frac{2500}{3667} = 1.682$	$1 + \frac{2500}{3667} = 1.682$	$1 + \frac{3550}{5196} = 1.683$	$1 + \frac{3550}{5196} = 1.683$	
u	I	16700	16700	20200	20200	20200	20200	
	II	2230	2230	21000	21000	21300	21300	
u_1	I	24000	24000	24000	24000	24000	24000	
	II	3170	3770	25800	26200	26200	26300	
E		− 1790	− 1790					
u_e		7100	7100					

REPORT ON QUEBEC BRIDGE.

APPENDIX B

SUSPENDED SPAN:-

	Vertical Posts.		Main Diagonals.					
Member-	L_2-U_2	L_4-U_4	U_0-C_1	C_1-L_2	U_2-C_3	C_3-L_4	U_4-C_5	C_5-L_6
A	134 gr. 119 net	78 gr. 70 net	204	191	118	114	75 net	94 gr. 82 net
$\frac{l}{r}$	$\frac{748}{10.7} = 70$	$\frac{792}{11.6} = 68$						$\frac{1015}{11.2} = 91$
D	− 1215	− 300	+ 3085	+ 2840	+ 1495	+ 1285	+ 445	+ 215
L	− 646	− 373	+ 1433	+ 1365	+ 834	+ 813	+ 509	+ 494
	+ 95	+ 235	−	− 31	− 106	− 196	− 282	− 387
S	− 100	− 15	+ 285	+ 265	+ 145	+ 120	+ 45	+ 20
W	0	0	0	0	0	0	0	0
D+L+S	− 1961	− 688	+ 4803	+ 4470	+ 2474	+ 2218	+ 999	+ 729
D+1½L+S+⅓W	− 2284	− 874	+ 5519	+ 5152	+ 2891	+ 2624	+ 1253	+ 976
φ	$1+\frac{1120}{1956}=1.572$	$1+\frac{65}{908}=1.072$	$1+\frac{3085}{4518}=1.683$	$1+\frac{2809}{4236}=1.663$	$1+\frac{1389}{2435}=1.570$	$1+\frac{1089}{2294}=1.475$	$1+\frac{163}{1236}=1.132$	$\frac{387}{1096}=0.353$
u I	13400	9200	20200	20000	18800	17700	13600	4240
u II	14600	8800	23500	23400	21000	19400	13300	8900
u_1 I	17000	17200	24000	24000	24000	24000	24000	24000
u_1 II	17000	11200	27000	27000	24500	23000	16700	11900
E	− 540		+ 2155	+ 1870	+ 535			
u_e	[illegible]		10600	9800	4500			

REPORT ON QUEBEC BRIDGE

APPENDIX B

SUSPENDED SPAN

Member	Suspenders: Loa-Uo	Suspenders: L1-C1	Suspenders: L3-C3	Suspenders: L5-C5	Diagonal Substruts: Loa-C1	Diagonal Substruts: L2-C3	Diagonal Substruts: L4-C5
A	50	42 net	42 net	42 net	49 gr.	53 gr.	53 gr.
$\frac{l}{r}$					$\frac{828}{10.15} = 82$	$\frac{969}{11.55} = 84$	$\frac{1016}{11.55} = 88$
D	+ 340	+ 265	+ 255	+ 275	− 260	− 210	− 230
L	+ 471	+ 405	+ 458	+ 458	− 292	− 280	− 265
	− 17				+ 24	+ 26	+ 21
S	+ 40	+ 40	+ 45	+ 45	− 25	− 25	− 25
W	0	0	0	0	0	0	0
D + L + S	+ 851	+ 710	+ 758	+ 778	− 577	− 515	− 520
D + $1\frac{1}{2}$L + S + $\frac{1}{3}$W	+ 1086	+ 912	+ 987	+ 1007	− 723	− 655	− 652
φ	$1+\frac{323}{828} = 1.390$	$1+\frac{265}{670} = 1.396$	$1+\frac{255}{713} = 1.358$	$1+\frac{275}{733} = 1.375$	$1+\frac{236}{376} = 1.410$	$1+\frac{184}{516} = 1.357$	$1+\frac{209}{516} = 1.405$
u I	16700	16800	16300	16500	13500	12700	13100
u II	17000	16900	18000	18500	11800	9700	9800
u_1 I	24000	24000	24000	24000	15800	15600	15200
u_1 II	21700	21700	23500	24000	14800	12400	12300
E	+ 340	+ 205	+ 110		− 305	− 535	
u_e	6800	4900	2600		6200	10100	

Report on Quebec Bridge. Appendix B.

Table of Permissible Unit Strains for Compression Members.

p = permissible strain in lbs. pr. sq. in.

l - unsupported length of member in inches.

r = least radius of gyration in inches.

$$p = 21000 - 90\frac{l}{r}$$

$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p
30	18300	51	16410	72	14520	93	12630
31	18210	52	16320	73	14430	94	12540
32	18120	53	16230	74	14340	95	12450
33	18030	54	16140	75	14250	96	12360
34	17940	55	16050	76	14160	97	12270
35	17850	56	15960	77	14070	98	12180
36	17760	57	15870	78	13980	99	12090
37	17670	58	15780	79	13890	100	12000
38	17580	59	15690	80	13800	101	11910
39	17490	60	15600	81	13710	102	11820
40	17400	61	15510	82	13620	103	11730
41	17310	62	15420	83	13530	104	11640
42	17220	63	15330	84	13440	105	11550
43	17130	64	15240	85	13350	106	11460
44	17040	65	15150	86	13260	107	11370
45	16950	66	15060	87	13170	108	11280
46	16860	67	14970	88	13080	109	11190
47	16770	68	14880	89	12990	110	11100
48	16680	69	14790	90	12900	111	11010
49	16590	70	14700	91	12810	112	10920
50	16500	71	14610	92	12720	113	10830

$$p = 24000 - 100\frac{l}{r}$$

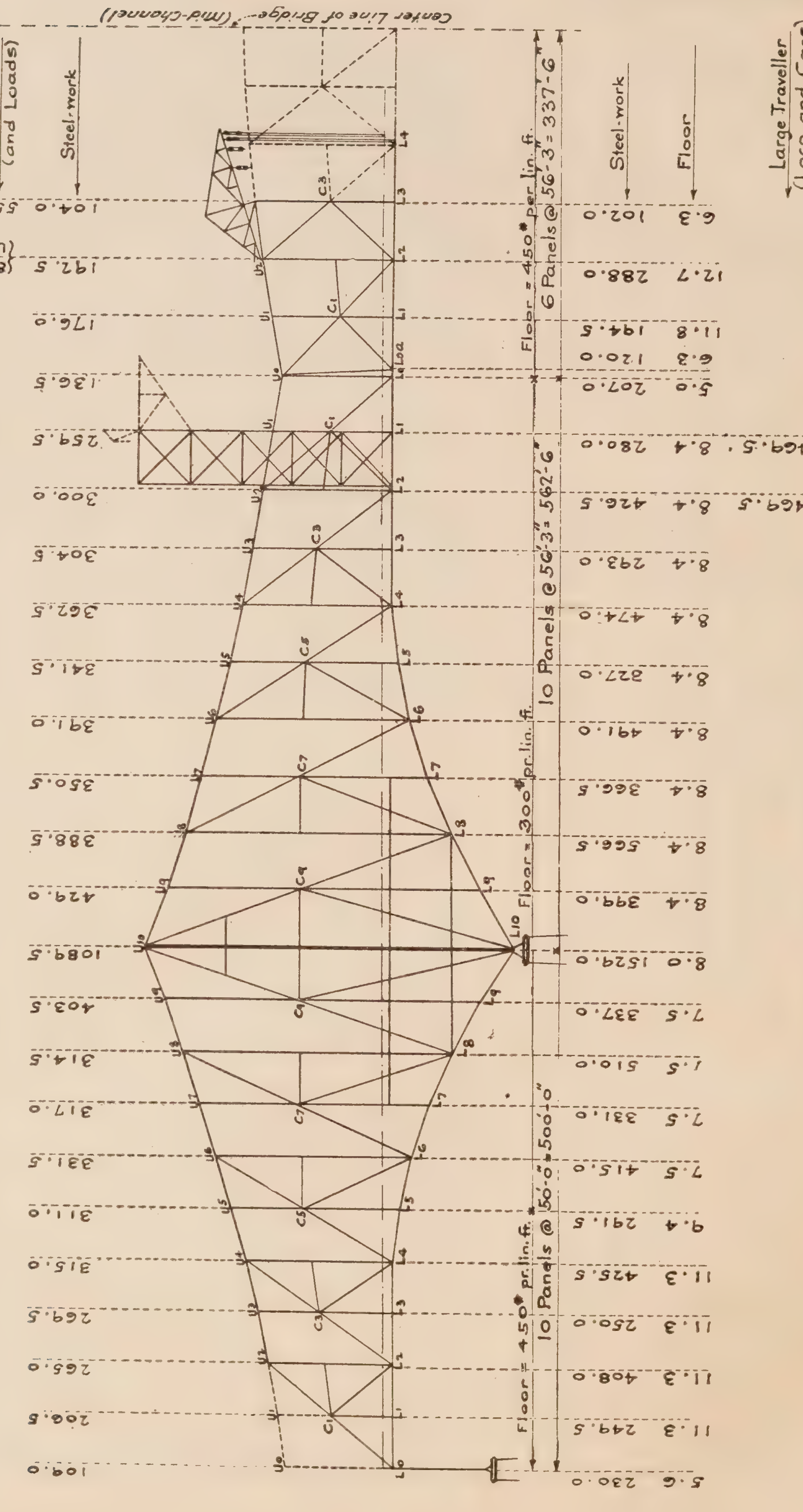
$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p
30	21000	51	18900	72	16800	93	14700
31	20900	52	18800	73	16700	94	14600
32	20800	53	18700	74	16600	95	14500
33	20700	54	18600	75	16500	96	14400
34	20600	55	18500	76	16400	97	14300
35	20500	56	18400	77	16300	98	14200
36	20400	57	18300	78	16200	99	14100
37	20300	58	18200	79	16100	100	14000
38	20200	59	18100	80	16000	101	13900
39	20100	60	18000	81	15900	102	13800
40	20000	61	17900	82	15800	103	13700
41	19900	62	17800	83	15700	104	13600
42	19800	63	17700	84	15600	105	13500
43	19700	64	17600	85	15500	106	13400
44	19600	65	17500	86	15400	107	13300
45	19500	66	17400	87	15300	108	13200
46	19400	67	17300	88	15200	109	13100
47	19300	68	17200	89	15100	110	13000
48	19200	69	17100	90	15000	111	12900
49	19100	70	17000	91	14900	112	12800
50	19000	71	16900	92	14800	113	12700

APPENDIX B.

REPORT ON QUEBEC BRIDGE.

CONDITIONS ON AUG. 29, 1907,

Showing Concentrations on Panel Points produced by Dead Load and Travellers.



Loads are for one Truss and are given in units of 1000#

REPORT ON DESIGN OF QUEBEC BRIDGE BY C. C. SCHNEIDER.

APPENDIX C.

THEORY OF COLUMNS.

A REVIEW OF EXISTING LITERATURE AND EXPERIMENTS.

An ideal column with a straight axis and of uniform material, loaded in the direction of its axis, would fail in direct compression by crushing. In practice, a column will fail by buckling caused by lateral deflection.

Failure by direct compression or tension is caused by strains which exceed the resistance of the material. Since these strains are in direct proportion to the loads causing them, it became customary to measure the safety of a structure by the ratio of the working strain to the ultimate strength, instead of by the ratio of the permissible load to the load causing failure.

Failure by buckling, however, is not necessarily the result of overstraining the material, as the strains are not in direct proportion to the corresponding loads (see examples page 192), but depend upon certain conditions which influence the strength of a column considered as a member of a structure.

Perhaps the clearest conception of buckling can be obtained by considering it as the result of unstable equilibrium between the external and internal forces. Assuming a steel spring (Fig. 1) rigidly fixed at the bottom, and loaded at the top with a weight W, then the spring will slightly deflect laterally, but will remain in equilibrium. If W is gradually increased, a condition will be reached where equilibrium is no more possible, and the weight will drop suddenly. The spring has lost its supporting power at this moment of unstability, but the weight may go to the bottom without producing any excessive strains in the spring.

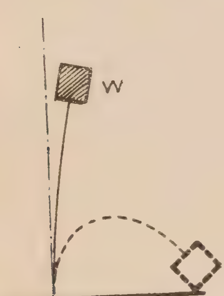


Fig. 1

The lateral deflection of a column is caused by an initial eccentricity as the load will not be exactly in the center, nor the axis be mathematically straight and the material uniform throughout the column, owing to irregularities in rolling, or caused by straightening, riveting, drifting, &c. (In an I-beam 8 feet long, Bauschinger found a variation of 5 per cent. in the elastic modulus and in the ultimate strength.)

This initial eccentricity and the deflection produced by it will cause bending and shearing strains in the column in addition to direct compression.

The *average* compressive strain obtained by dividing the buckling load—that is, the load under which the column fails—by the area of its section is called the *buckling strain.*

I. Long Columns.

In order to find a formula for the buckling strain, long columns which fail with a buckling strain within the elastic limit will first be considered. To apply to these the theory of elasticity is not strictly correct, as the maximum fibre strain may have exceeded the elastic limit; however, this, as will be shown later, affects the buckling load only very slightly. The true elastic limit for wrought iron and steel is almost identical with the limit of proportionality between strain and deformation.

Let it be assumed that an elastic column with hinged ends free to move in the direction of its original axis, and subjected to an axial load P, has been deflected laterally (see fig 2). Neglecting the shortening of the column and the influence of the shearing strains, and assuming $s=x$, the elastic line is represented by the differential equation

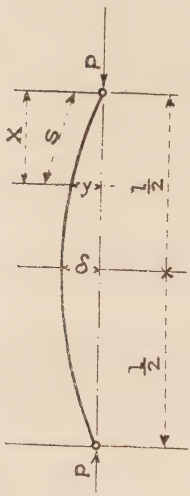


Fig. 2

$$\frac{d^2 y}{dx^2} = \frac{M}{EI} \quad \ldots \ldots \quad (1)$$

where bending moment $M=Py$, $I=$ Moment of Inertia of the section, and $E=$ Modulus of Elasticity of the material of the column.

Twice integrating,

$$\text{then } y = \delta \sin x \sqrt{\frac{P}{EI}} \quad \ldots \ldots \quad (2)$$

where $\delta=$ deflection at the centre.

The elastic line, therefore, is a sinus curve for $x=l$, and $y=o$, then from equation (2)

$$P_0 = \pi^2 \frac{EI}{l^2} \quad \ldots \ldots \quad (3)$$

as the load which holds the internal strains in equilibrium.

This formula is known as Euler's formula, having been first introduced by Euler in 1759. Since this formula does not contain δ, P_0 is the load which, after a lateral deflection is once started, may increase this deflection, and with it the fibre strain, rapidly and finally produce buckling. This buckling load, therefore, is independent of the strength of the material as long as E remains the same.

According to Euler's formula, a column made of steel containing 3 per cent nickel, with an ultimate strength about 50 per cent higher than ordinary carbon steel, could safely carry a load only about 4 per cent greater than an identical column made of ordinary carbon steel; that is, in proportion of the moduli of elasticity.

On account of the assumptions made in deriving formula (3), P_0 does not correctly represent the buckling load. More correct formulae have been derived by Grashof, (Festigkeits Lehre, published 1866) who gives

$$P = \frac{\pi^2 EI}{l^2}\left(1 + \frac{\pi^2 \delta^2}{8 l^2}\right) \quad \ldots \ldots \quad (4)$$

and by Wm. Cain, (Trans. A. S. C. E., Vol. XXXIX.) who derives

$$\delta^2 = 16\left[\frac{l}{\pi}\sqrt{\frac{EI}{P}} - \frac{EI}{P}\right] \quad \ldots \ldots \quad (5)$$

An investigation of formulae (4) and (5) shows that if P exceeds $P_0 = \frac{\pi^2 EI}{l^2}$, a certain deflection δ corresponds to the load P; but that a very small increase over P_0 is sufficient to make the deflection excessive and cause failure; so that P_0 can practically be regarded as the buckling load. In these formulæ for $\delta = o$, $P = P_0$, in Euler's formula; in other words, P_0 represents the load under which bending just begins, so that for smaller loads than P_0 the compressive strains are uniformly distributed over the section.

In formulae (3), (4) and (5), the initial eccentricity 'e' (fig. 3) has been assumed negligible as compared with the deflection δ. Investigation of the formula given on page 191 for eccentric loading shows that any load P, even below P_0, can produce deflection; but if the eccentricity 'e' is small, the buckling load will be only slightly smaller than P_0, although the maximum fibre strain produced thereby may be higher than the buckling strain. This is another reason for regarding P_0 as the actual buckling load. A greater initial eccentricity will reduce the buckling load by giving fibre strains above the limits of safety.

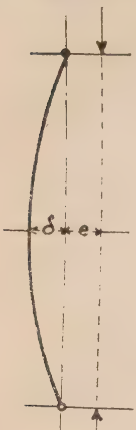


Fig. 3

On this basis, many attempts have been made to derive formulae giving the load which would cause failure by excessive fibre strains. (See J. M. Moncrieff, Trans. A. S. C. E., Vol., XLV.)

The yield point must be regarded as the highest safe fibre strain, because as soon as it is exceeded the deflection increases rapidly until finally failure occurs.

On the other hand, an initial bend in the column can counteract the initial eccentricity of the load, keeping the column in stable equilibrium even for a greater load than P_0. These cumulative influences explain the different actions of columns in testing as regards deflections and breaking loads.

As it is impossible to determine for every case the initial eccentricity, a buckling formula has to be derived for the case of an ideal, or nearly ideal column; provided this formula agrees with the results of experiments made under conditions as nearly as possible like those of the ideal column.

In determining the safe working load, the lowest test result should be used with a margin of safety.

Similar conditions occur in bending. The permissible strain for bending is derived from the ultimate strength with the provision that under the worst condition the fibre strain shall remain below the yield point.

Column tests, especially those with point bearings made by Tetmajer and Bauschinger, prove that for long columns, which fail with a buckling strain within the elastic limit, Euler's formula gives correct results. (See L. v. Tetmajer, 'Die Gesetze der Knickungs festigkeit,' 3rd edition, Leipzig and Wien, 1903, also, 'Mitteilungen der Materiel Prüfungsanstalt,' München, 1887, by Bauschinger.)

Euler's formula (3) does not give the greatest strains actually existing in a column. This has caused the introduction of various formulæ which apparently express the relation between the load and the corresponding greatest strain. Since, however, as has been seen, strains in buckling are very uncertain, all the formulæ based on strains contain one or more coefficients, the values of which have to be derived empirically from the buckling load of column tests. Dividing the buckling strain thus found by a factor of safety, the formulæ represent more or less correctly safe loads, but they do not give the actual safe unit strains.

One of these is the extensively used 'Rankine Formula.'

$$k_0 = \frac{k_u}{1 + c\,\dfrac{l^2}{r^2}} \qquad \qquad (6)$$

where k_0 = buckling strain, k_u an assumed constant approximately equal to the yield point and c a constant to be derived from tests. It has been proven, however, by experiments and analytically, that c is not constant but varies not only with the material, but also with the value of $\frac{l}{r}$ and with the average unit strain. Tetmajer found by tests a variation of $c = 0{\cdot}000448$ to $0{\cdot}000136$ for wrought iron, and $c = 0{\cdot}000370$ to $0{\cdot}000130$ for steel.

The fact that it is possible to give k_u and c such values that k_0 corresponds fairly well with observed buckling strains within the practical limits of $\frac{l}{r}$ makes the formula applicable to practical use. It thus becomes an empirical formula.

Dividing P_0 by the area, Euler's formula takes the following form:—

$$k_0 = \pi^2 E \frac{r^2}{l^2} \quad \ldots \ldots \ldots \quad (7)$$

where k_0 represents the buckling strain. Giving k_0 the value of the elastic limit, and solving for $\frac{l}{r}$, the limit for Euler's formula is found to be

$$\frac{l}{r} = \pi \sqrt{\frac{E}{k_0}}$$

Tetmajer found the following values:—

For wrought iron with an elastic limit $= 22{,}600$, $\frac{l}{r} = 112$

soft steel " " $= 27{,}100$, $\frac{l}{r} = 105$

medium steel " " $= 28{,}400$, $\frac{l}{r} = 105$

and with $E = 28{,}450{,}000$, $30{,}580{,}000$ and $32{,}000{,}000$ respectively, Euler's formula becomes

$k_0 = 280{,}800{,}000 \left(\frac{r}{l}\right)^2$ for wrought iron,

$k_0 = 301{,}800{,}000 \left(\frac{r}{l}\right)^2$ " soft steel,

$k_0 = 315{,}900{,}000 \left(\frac{r}{l}\right)^2$ " medium steel.

II. Short Columns.

Thus far this subject has been considered theoretically only, in order to give a clear account of the nature of buckling. Columns which fail with buckling strains above the elastic limit will now be considered. These include the majority of cases occurring in actual practice.

Since, as shown above, Euler's formula is limited, and is applicable to steel columns only whose $\frac{l}{r}$ exceeds 105, it appears desirable to consider the subject wholly from a practical standpoint and endeavour to find an empirical buckling formula based on experiments.

The first question to be considered is: What is the buckling strain for a very short column (theoretically $\frac{l}{r} = o$)?

Fig. 4 represents the typical deformation diagram for wrought iron or steel, the abscissas ϵ representing the elongations corresponding to the stresses k as ordinates. P denotes the limit of proportionality, or elastic limit, and Y the yield point. Up to the elastic limit, the modulus of elasticity E for wrought iron and steel is constant, but is variable for higher strains. If the values of E for strains above the elastic limit

were known and applied to Euler's formula, it would still express correctly the buckling load.

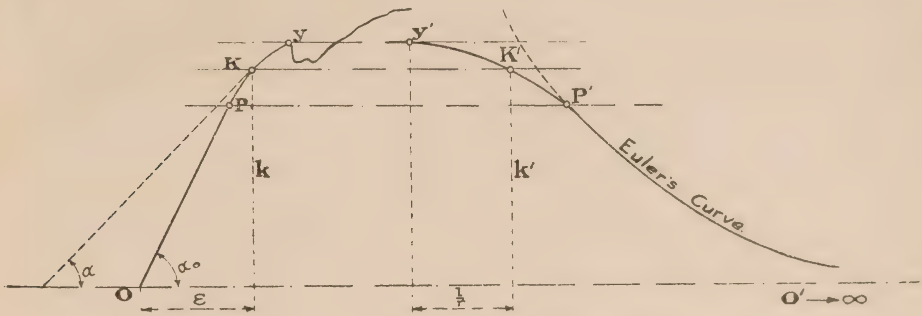


Fig. 4 Fig. 5

Drawing a tangent to the curve at a point K, the corresponding modulus of elasticity may be represented by

$$E = \frac{d\,k}{d\,\epsilon} = tg.\,\alpha;$$

introducing this in Euler's formula and solving for $\frac{l}{r}$

$$\text{then } \frac{l}{r} = \pi \sqrt{\frac{tg.\,\alpha}{k}}$$

This equation enables us to construct the curve of Fig. 5, where the abscissas represent the values $\frac{l}{r}$ and the ordinates the strains k. If point K travels on the straight line from O to P, E is constant $= E_0 = tg\ \alpha_0$ and point K' follows Euler's curve from O' to P', the corresponding values of $\frac{l}{r}$ for point P' being those given on page 186.

If point K continues from P to Y, $tg\ \alpha$ gradually decreases from $tg\ \alpha_0$ to zero, while point K' travels over curve $P'\ Y'$ and $\frac{l}{r}$ gradually becomes zero.

This means that a very short column becomes unstable when the buckling strain reaches the yield point, since this is the point of first horizontal tangency. As is well known, the yield point, commercially called elastic limit, manifests itself in testing by the sudden drop of the test load.

Cast iron does not follow the law of proportionality, nor has it a yield point (see deformation diagram, fig. 6).

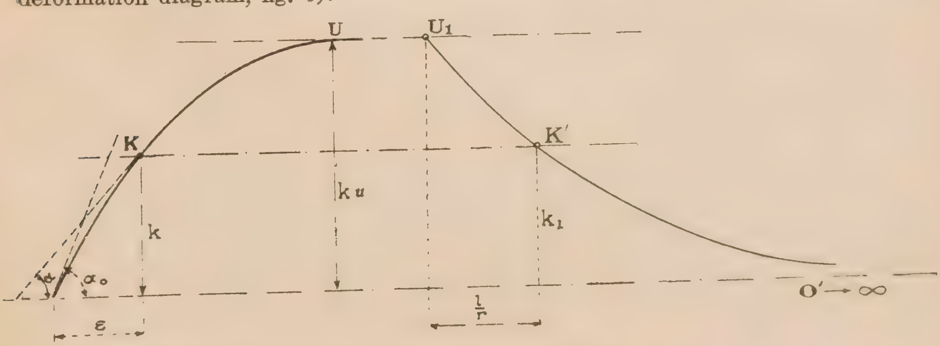


Fig. 6 Fig. 7

$tg\ \alpha$ decreases from the point of zero strain, and becomes zero where the tangent to the deformation curve becomes horizontal; that is, at the point U of ultimate strength k_u. Point K' does not follow Euler's curve (fig. 7) but reaches U_1 for $\frac{l}{r} = O$ in a more regular parabolic curve. The buckling strain for very short columns $\left(\frac{l}{r} = O\right)$ is therefore equal to the ultimate strength. This explains the fact that short cast iron columns show a much higher resistance to buckling than wrought iron or ordinary steel columns.

If tests were made with short columns of very hard steel, in which the yield point and ultimate strength are close together, these tests would evidently also show a proportionately greater buckling strength than those of ordinary steel.

While some engineers are of the opinion that the ultimate strength in tension should be regarded as the buckling strain for $\frac{l}{r} = O$, others have recognized the yield point as this ultimate buckling strain. (See J. B. Johnson's 'Modern Framed Structures,' p. 159.)

What is generally called yield point (about 60 to 70 per cent of the ultimate strength for steel) is an apparent strain obtained from tension tests based on the original area of the bar. Since the area of the bar has decreased, the true yield point must be higher, and this is equal to the true yield point in compression. The apparent yield point in compression based on the original area of the compression member is still higher since in compression the area has increased and this yield point must be regarded as ultimate buckling strain, because the latter is also based on the original area.

Since the increase of the area is not known, the ultimate buckling strain must be found from tests. Undoubtedly a column of say $\frac{l}{r} = 5$ in the testing machine acts practically the same as one of $\frac{l}{r} = O$; that is, the strain is uniformly distributed up to the breaking point, since any accidental eccentricity would cause only very small bending strains. The buckling strains thus found can, therefore, be considered as the ultimate buckling strain for $\frac{l}{r} = O$.

Tetmajer found for this strain which he calls 'a kind of compressive strength, different from, but comparable to the crushing strength of cubes,' the following values:

	Lbs. per sq. in.
For wrought iron.	$k_u = 43{,}100$
" soft steel.	$k_u = 44{,}100$
" medium steel.	$k_u = 45{,}700$

A rational column formula should contain these values as the limiting buckling strain for $\frac{l}{r} = O$ and give buckling strains decreasing from this limit with increasing $\frac{l}{r}$.

The curve representing this formula should, moreover, intersect Euler's curve at the point for which k_0 is equal to the true elastic limit. As this latter strain as well as the yield point is more or less variable, even in the same material, it is evident that points P' and Y' (fig. 5) can be chosen within certain limits. Owing to the greatly varying test results, it is also evident that a great number of different curves can be drawn between points P' and Y' as representing approximately the average of the plotted test results.

For all practical purposes, the simplest curve is naturally the best and that is the straight line.

The writer considers that all these more or less complicated analytical formulæ (like Rankie's, &c.) are not justified. Analytical formulæ based on the theory of proportionality between the strain and deformation (with a constant E) cease to be correct for the buckling strains which are now being considered; and have been made applicable to the latter by merely choosing empirical coefficients.

The following publications contain the results of column tests and diagrams, with the results plotted and the different curves representing the formulæ. An examination will show that the straight line fits at least as well as any curve:—

1.—L. F. G. Bouscaren. Trans. A.S.C.E., Vol. IX.
2.—J. Christie, 'Experiments on the Strength of Wrought Iron Struts.' Trans. A.S.C.E., Vol. XIII.
3.—T. H. Johnson, 'On the Strength of Columns.' Trans. A.S.C.E., Vol. XV.
4.—C. A. Marshall. Trans. A.S.C.E., Vol. XVII.
5.—C. L. Strobel, 'Experiments upon Z-Iron Columns.' Trans. A.S.C.E., Vol. XVIII.
6.—Tests of Metals made at Watertown Arsenal. Vols. 1881, 1882, 1883, 1884 and 1885.
7.—A. Marston, 'On the Theory of the Ideal Column.' Trans. A.S.C.E., Vol. XXXIX.
8.—J. M. Moncrieff, 'The Practical Column.' Trans. A.S.C.E., Vol. XLV.
9.—Johnson, Bryan and Turneaure, 'The Modern Framed Structures.' 8th Edition, page 168.
10.—G. Lanza, 'Applied Mechanics,' page 416.
11.—L. v. Tetmajer, 'Die Gesetze der Knickungs festigkeit.' 3rd Edition, 1903.
12.—Prof. Bauschinger, 'Mitteilungen der Material prüfungsanthalt München.' 15th Vol.

The straight line formula

$$k_o = k_u - c\frac{l}{r} \quad \ldots \ldots \ldots \ldots \quad (8)$$

was first proposed in 1886 by T. H. Johnson (see Trans. A.S.C.E., Vol. XV.) and is now generally used. He derived it from tests of wrought and cast iron and steel columns made by Hodgkinson, Christie and others under greatly varying conditions, and proposed for columns with round ends, the following buckling strength:—

Wrought iron, $42{,}000 - 203\frac{l}{r}$, upper limit $\frac{l}{r} = 138$

Carbon 0·12%, soft steel, $52{,}500 - 284\frac{l}{r}$, " $\frac{l}{r} = 123$

" 0·36%, hard steel, $80{,}000 - 534\frac{l}{r}$, " $\frac{l}{r} = 100$

They represent straight lines drawn from k_u tangent to Euler's curve. By referring to the above-mentioned tests, it is evident that k_u is too high for steel, while the point of meeting Euler's curve is too low. A less inclined line, taking the former point lower and the latter higher would give more correct results.

Based on his own numerous tests of wrought iron and steel columns with point bearings, L. v. Tetmajer introduced a stright line formula, at the same time proving the correctness of Euler's formula for buckling strains lower than the elastic limit.

(See 'Mitteilungen der Material Prüfungsanstalt, Zürich,' Vol. VIII, also L. v. Tetmajer, 'Die Gesetze der Knickungs festigkeit,' 3rd part, 1903.) He proposed for

$$\text{Wrought iron, } k_o = 43{,}100 - 183\,\frac{l}{r},\quad \frac{l}{r} \lesseqgtr 112$$

$$\text{Ultimate strength} < 57{,}000,\text{ soft steel,}\quad k_o = 44{,}100 - 162\,\frac{l}{r},\quad \frac{l}{r} \lesseqgtr 105.$$

$$\text{“}\quad > 57{,}000,\text{ medium steel,}\quad k_o = 45{,}700 - 165\,\frac{l}{r},\quad \frac{l}{r} \lesseqgtr 105$$

Since steel columns with $\frac{l}{r} > 105$ are used for unimportant parts only, and the difference between Euler's and the straight line formula is only small for $\frac{l}{r}$ from 105 to 120 (which is generally the practical limit), it is justifiable to use the straight line formula throughout.

The permissible unit strain for tension is usually deduced from the ultimate strength; while that for compression must be deduced from the considerably lower buckling strain. For compression therefore, a smaller factor of safety is permissible than for tension, since the strains in either case must remaiin with a margin of safety below the true yield point.

If, in accordance with usual practice, a unit strain of 16,000 pounds per square inch in tension for structural steel (55,000 to 65,000 ultimate strength) is used,the same strain is permissible for a column with $\frac{l}{r} = O$ in compression. For longer columns, this strain has to be reduced by the formula in order to have the same factor of safety for all ratios of $\frac{l}{r}$. The formula for the permissible unit strain

$$s_o = 16000 - 70\,\frac{l}{r} \qquad (9)$$

which was adopted by the Committee on Steel Structures of the American Railway Engineering and Maintenance of Way Association will allow a safety of about 3.

Thus far only the case of a column with ends free to rotate has been considered. This case, however, does not occur in practice; the ends will always offer more or less resistance to turning. All cases, however, can be treated similarly by assuming the so-called buckling length; that is, the distance between points of contraflexure.

The assumption of the buckling length is mainly a matter of practical judgment, since in practice no column will correspond either to theory or to experiments.

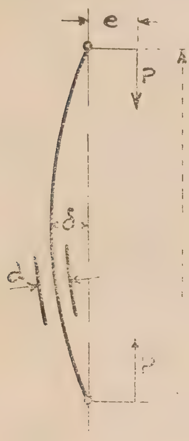
For compression members with hinged ends, the friction of the hinges should be entirely neglected and even for compression members with riveted, and, therefore, partly fixed ends, the free buckling length should not be assumed less than the distance between connections on account of the secondary strains due to the elastic deformation of the truss. These secondary strains, as will be seen from Appendix D, are the result of bending moments which may partly or entirely counteract the fixity of the ends.

III. The Eccentrically Loaded Column.

Since in practice a column is always more or less eccentrically loaded, this case must be considered in order to determine to what degree an eccentricity can affect the buckling load of the ideal column. This will also show the increase of the fibre strains when the load increases. Of course, only comparatively small eccentricities are considered; such as may occur in compression members of trusses.

In a column loaded eccentrically and parallel to the original axis, the deflection δ can be accurately determined; hence also the bending moments and the fibre strains, provided the latter do not exceed the limit of proportionality. In order to get comparative figures, however, the following formula will be used for strains up to the yield point:—

With the notations of fig. 8, the extreme fibre strain k can be expressed by the well-known Navier formula



$$k = k_0 \left[1 + \frac{(e+\delta)\, d}{2\, r^2} \right] \quad \text{. . . . . (10)}$$

and the deflection by the formula

$$\delta = \frac{e}{\pi^2 \dfrac{E\, r^2}{k_0\, l^2} - 1} \quad \text{. . . . . (11)}$$

where 'e' is either an initial eccentricity or an initial bend and k_0 the load per square inch.

This formula shows that however small the eccentricity 'e' may be, the deflection δ will increase to excessive proportions and the column will fail absolutely when the denominator approaches zero.

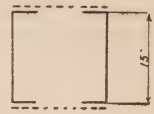
$$\text{But } \pi^2 \frac{E\, r^2}{k_0\, l^2} - 1 = O$$

is nothing else than Euler's formula, and it is seen at once that for very small 'e,' δ, and with it the fibre strain, becomes unsafe only in case the load approaches k_0 of Euler's formula.

Assuming that the column would fail when the maximum fibre strain reaches the yield point (according to Tetmajer's tests of eccentrically loaded columns this assumption is justified); that is, making k of equation (10) equal to the yield point and introducing the value of δ from formula (11) into equation (10), then an expression for the breaking load is found by solving for k_0.

A few examples, however, will better illustrate the relation between load and strains than the investigation of such a formula.

For all examples, a column composed of 15-in. ['s, 33 lbs. will be assumed with $r=5{\cdot}62$ in. and $d=15$ in. and $\pi^2E=300{,}000{,}000$.



$\frac{l}{r}$	Buckling strain for axial load.	Safe buckling strain, $16{,}000-70\frac{l}{r}$	Assumed eccentricity. e	Assumed load in lb. p. sq. in. k_o.	Deflection. δ	$e+\delta$	Bending strain k_c.	Maximum fibre strain k_u.
			In.		In.			
				7,600	0·06	0·16	300	7,900
				10,000	0·09	0·19	500	10,500
	$\pi^2E\left(\frac{r}{l}\right)^2$	....	0·1	15,000	0·25	0·35	1,200	16,200
				18,000	0·62	0·72	3,000	21,000
				20,000	2·5	2·6	9,500	29,500
				7,600	0·6	1·6	2,900	10,500
				10,000	0·9	1·9	4,500	14,500
120	20,800	7,600	1·0	15,000	2·5	3·5	12,500	27,500
				17,700	5·5	6·5	27,300	45,000
				7,600	2·9	7·9	14,200	21,800
			5·0	10,000	4·6	9·6	22,700	32,700
				12,000	6·8	11·8	33,500	45,500
				10,400	0·03	0·13	300	10,700
	$45{,}000-160\frac{l}{r}$	....	0·1	20,000	0·07	0·17	800	20,800
				30,000	0·18	0·28	2,000	32,000
				10,400	0·29	1·29	3,200	13,200
80	32,200	10,400	1·0	20,000	0·74	1·74	8,200	28,200
				28,000	1·47	2·47	16,500	44,500
				10,400	1·45	6·45	15,800	25,800
			5·0	16,500	2·7	7·7	30,000	46,500
				13,200	0·08	1·08	3,300	16,500
	$45{,}000-160\frac{l}{r}$	....	1·0	20,000	0·12	1·12	5,300	25,300
				30,000	0·19	1·19	8,500	38,500
				13,200	0·38	5·38	16,800	30,000
40	38,600	13,200	5·0	20,000	0·60	5·60	26,500	46,500

The loads underlined are approximately the buckling strains caused by excessive fibre strains.

Applying the foregoing to a straight column apparently centrally loaded, it is seen at once that its safety cannot be judged by merely comparing the working load (including impact, if any) with the buckling load, but that also the possibility of an eccentricity must be considered, since under unfavourable conditions the maximum fibre strains may become excessive under the working load. It is, however, not necessary to keep these strains within the same limits as allowed for tension or direct compression, but is sufficient if they remain within the yield point, since they are only accidental.

In this respect, columns differ from beams or tension members, as for these load and strain are in direct proportion so that only the one condition has to be fulfilled to keep the working strain, under the most unfavourable condition, within the yield point.

What should be considered as the most unfavourable condition as to eccentricity is a matter of judgment. But from the foregoing examples, it is evident that for columns of lengths such as used in practice there is sufficient safety against excessive accidental fibre strains when using for static loads the permissible unit strain given

by the formula $16{,}000 - 70 \frac{l}{r}$, since the eccentricities which would cause excessive fibre strains under the working load are evidently greater than those likely to occur in good practice.

It must be remembered that the column with frictionless hinged ends is considered here. In practice more or less fixity of the ends counteracts the influence of a possible eccentricity; that is, the free buckling length will be reduced, unless the eccentricity is excessive, or secondary strains are likely to occur.

In good practice, the latter cases should be carefully considered and, if found of importance, special provision should be made in designing the column.

The writer has endeavoured to treat this subject merely from a practical standpoint, applying theory only so far as necessary to explain some fundamental principles, as the many elaborate theories advanced on this subject have been productive of more or less confusion.

Considering that static computations are only approximations in any case, the writer is of the opinion that our knowledge of the behaviour of compression members under strain is sufficient to enable us to design columns with as much approach to accuracy as any other member of a structure subject to bending. Additional tests on large columns, corresponding to those used in modern practice, made under the supervision of experienced experimenters, would tend to further reduce the factor of ignorance on this subject.

THE DESIGN OF LATTICING OF COLUMNS.

If a column is made up of several shapes or parts, they have to be connected in such a manner that they will act as a unit. In an ideal column each part would take its share of the load and no connection would be required. In practice, however, as stated before, bending will occur before the buckling load is reached, causing shearing strains which have to be transferred through the connections, as latticing, tie plates or cover plates. These connection parts have, therefore, to perform the same function as the web of a girder or the web system of a truss. It has also been previously explained that, due to the variety of causes producing an initial eccentricity, it is not possible to figure exactly the bending strains caused by a given load, not even at the time of breaking. And, since the shearing strains depend on the bending strains, the same uncertainty applies to these. The design of latticing, therefore, will remain largely a matter of practical judgment like the design of other details, until by means of numerous comparative tests, an empirical basis can be established.

There is, however, a rational method of dimensioning latticing analytically, which agrees well with actual examples found in existing bridges of usual dimensions.

When a column is bending the maximum fibre strain will exceed the average buckling strain, the difference being the bending strain. As a very short column theoretically $\frac{l}{r} = O$) will fail when the *average* buckling strain has reached the yield point, while a longer column whose *maximum* fibre strain has reached the yield point, will deflect rapidly and fail under a small increase of the load, it is reasonable to assume that a column will fail by buckling when the maximum fibre strain reaches the yield point; in other words, when the bending strain is equal to the difference between the yield point and the buckling strain.

Extremely long columns which may buckle without their fibre strain reaching the yield point (see example given for a steel spring) are not used in structural work and are, therefore, beyond the scope of this investigation.

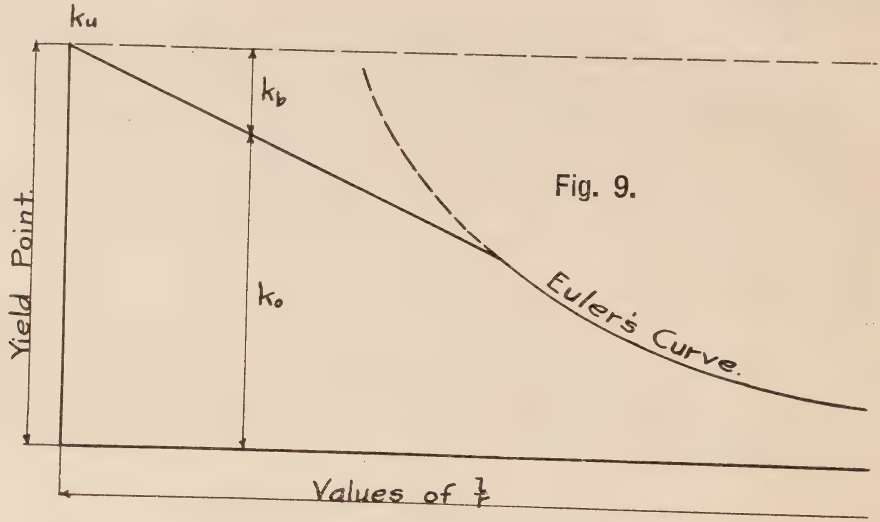


Fig. 9.

In the straight line formula $k_o = k_u - c \dfrac{l}{r}$, the bending strain is therefore, $k_b = c \dfrac{l}{r}$, represented in the diagram of buckling strains (fig. 9) by the ordinates between the buckling curve k_o and the horizontal line through the yield point k_u.

It is evident that every part of the column must be able to resist the bending corresponding to the strain k_b, as otherwise its full strength would not be developed.

Some lacing bars are in compression and others in tension. Those in compression must be treated in the same way as the column; using the same unit strain k_u, but reduced according to their $\dfrac{l}{r}$. Those in tension become ineffective when they stretch, as their elongation would permit a sudden increase in the deflection of the column and have, therefore, to be proportioned for the yield point in tension. A column thus proportioned has a uniform resistance against failure in all its parts, and if, instead of the respective yield points, the same permissible unit strains are used in proportioning column and latticing, a uniform safety is obtained for the column as a whole.

In order to find the shear due to the bending, the shape of the axis of the deflected column has to be assumed. As mentioned before, the elastic line of an axially loaded column is a sinus curve. If, however, the column has an initial eccentricity, the elastic line will approach a circular curve the more the greater the eccentricity compared to the resulting deflection. We will, therefore, assume the elastic line as a parabola which lies between the two curves. (Fig. 10.)

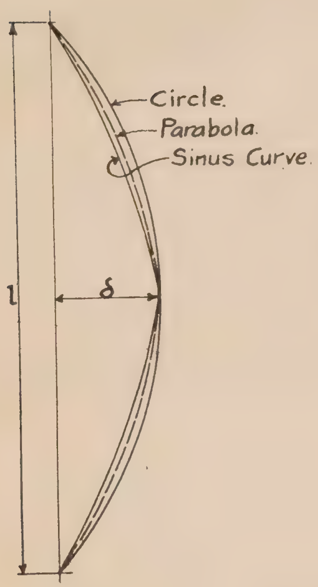


Fig. 10.

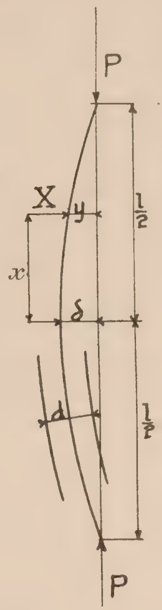


Fig. 11.

The equation of the elastic line with the notations taken from fig. 11 will then be

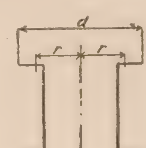
$$y = x^2 \frac{4\,\delta}{l^2} \text{ and } \frac{d\,y}{d\,x} = \frac{8\,\delta}{l^2}\,x$$

The maximum bending strain we have assumed as

$$k_b = c\,\frac{l}{r}\text{. which must be equal to} =$$

$$\frac{M \max}{R} = \frac{P\,\delta}{R}$$

where R = Moment of Resistence = $\frac{2\,ar^2}{d}$, a = area, r = radius of gyration, d = width of column.



Therefore we have

$$M \max = R\,c\,\frac{l}{r} = 2\,c\,\frac{ar}{d}\,l = P\,\delta$$

Since the bending moment at any point X is $M = Py$, the shear at the same point is

$$S = \frac{dM}{dx} = P\,\frac{dy}{dx} = \frac{8\,P\delta}{l^2}\,x$$

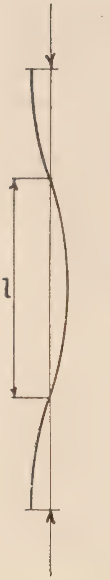
Substituting for $P\,\delta$ the value given above, we get

$$S = 16\,xc\,\frac{ar}{dl} \quad \ldots\ldots\ldots\ldots \quad (1)$$

and for $x = \frac{l}{2}$

$$S \max = 8\,c\,\frac{ar}{d} \quad \ldots\ldots\ldots\ldots \quad (2)$$

NOTE.—'r' is the radius of gyration laterally and d the width of the member, also laterally; that is, in the plane of the lacing. The 'a' is not the actual area used, but is the area required for the lateral radius of gyration and the corresponding l. In ordinary cases, however, the actual area can be used as 'a.'



From equation (1), it follows that the shear decreases toward the middle of the column. In practice, however, the ends are always more or less fixed so that the elastic line will take the shape shown in fig 12 or fig. 13, and l will be the distance between points of contraflexure.

Fig. 12. Fig. 13.

Since S max (according to equation (2)) is entirely independent of the length of the column, and since it occurs at the point of contraflexure, it follows, that it may occur at almost any point. The latticing should, therefore, be proportioned for the maximum shear throughout the entire length of the column.

The following gives the proportioning of various systems of column latticing:—

Fig. 14.

(1) If the column consists of two segments (fig. 14) connected by one system of single lacing bars, the shear S (under which we will now always understand S max) has to be taken up by one bar. The required area A of the bar is

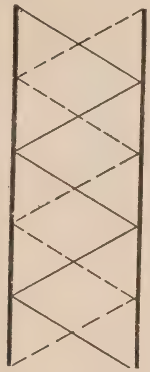
$$A = \frac{S}{k} \sec a \text{ and since } S = 8\,c\,\frac{ar}{d}$$

$$A = 8\,\frac{c}{k}\;\frac{ar}{d} \sec a \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

k being the yield point in tension for a tension bar, and $k = k_u - c\,\frac{l}{r}$ for a compression bar; $\frac{c}{k}$ being a constant for the same bar, the size of the bar is a function of the properties of the column section only, and does not depend on the column length or on any strains. We can, therefore, in any given case, without knowing the column load and the permissible unit strain, judge if the latticing be sufficient for the section of the column. Thus we follow the accepted practice of designing the connections to develop the full strength of the member. Since the strains have only relative values, the permissible unit strains of 16,000 pounds for tension and $(16{,}000 - 70\,\frac{l}{r})$ for compression will be used hereafter instead of the final values of k given above.

Fig. 15.

Fig. 16.



If the system is double (fig. 15), or single and on two sides of the column (fig. 16), the area required in the bar is, of course, only half of that given by formula (3).

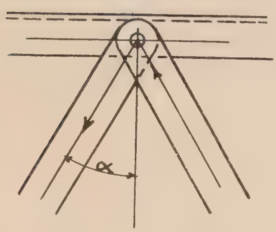
To find the number of rivets N required to connect the lattice bar, we have to remember that the allowed unit for shear is assumed $\frac{3}{4}$ of that in tension. If A_r = area of rivet, and A the net area required in the tension bar, we have

$$A = \frac{3}{4} N A_r, \text{ from which}$$

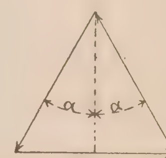
$$N = \frac{4}{3}\frac{A}{A_r} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

If the bars are connected by one rivet as in fig. 16, this rivet transmits the resultant of the two lattice strains. The strain on one bar is

Fig. 17.



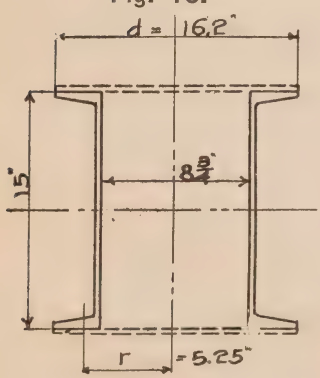
If the bars are connected by one rivet as in fig. 17, the rivet transmits the resultant of the two lattice strains.



The strain on one bar for single latticing on both sides is

$$S \frac{\sec a}{2}$$

Fig. 18.



Example:—

Column section 2 – 15-in. ['s 50 lbs. (fig. 18)

We will assume that the area 'a' required for buckling in either direction be the same: $a = 29{\cdot}4$ sq. in. and that the column shall have single lacing of $a = 30°$ on both sides.

$$\text{For } \frac{c}{k} = \frac{70}{16{,}000} = 0{\cdot}0044$$

since sec $a = 1{\cdot}16$, we find by formula (3) the net area of one bar.

$$A = \tfrac{1}{2} \times 8 \times 0{\cdot}0044 \, \frac{29{\cdot}4 \times 5{\cdot}25}{16{\cdot}2} \times 1{\cdot}16 = 0{\cdot}195 \text{ sq. in.}$$

1 bar $2\frac{1}{2} \times \frac{3}{8} = 0{\cdot}56$ square inch will be ample.

Number of $\frac{7}{8}$-in. rivets required $= N = \dfrac{4}{3} \; \dfrac{0\cdot195}{0\cdot6} = 0\cdot43$

per bar use one for 2 bars as in Fig. 17.

2.—*Columns with 3 Webs:* (Fig. 19.)

Fig. 19.

Required area of column $a = a' + 2a''$, where a' and a'' are the actual areas of the ribs reduced in proportion of the total required area 'a' to the actual area.

The longitudinal shear S' between two ribs for one panel length L has to be taken up by the diagonal $1-2$ of that panel.

The longitudinal shear per lineal inch is found from the transverse shear S by formula

$$t = \frac{SM}{I} = \frac{SM}{ar^2}$$

where M = Static Moment of the outer rib about the column axis $= a''e$, e being the distance of *centre of gravity* of the rib from the column axis.

't' of course decreases with S towards the point of maximum deflection, and S' could be found by integration for the length L. The error will, however, be small if we assume 't' constant for one panel length. We then get

$$S' = tL = \frac{SM}{ar^2} L = 8c \frac{ML}{dr} \left(\text{since } S = 8c \frac{ar}{d} \right) \quad . \; . \; . \; . \; . \; . \; . \; . \quad (5)$$

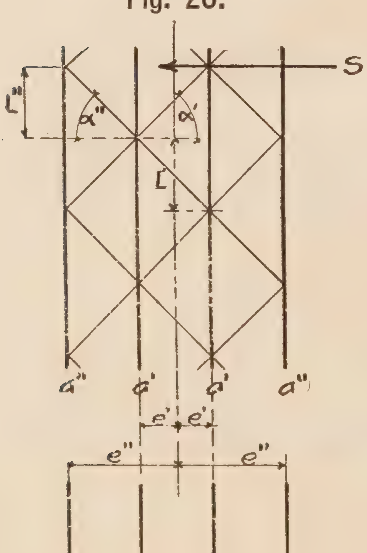
and the area of the bar required.

$$A = \frac{S'}{k} \operatorname{cosec} \alpha = 8 \frac{c}{k} \frac{ML}{dr} \operatorname{cosec} \alpha \, . \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (6)$$

or, $\frac{1}{2}$ of this if there are two sides of latticing.

3.—*Columns with 4 Webs:*

Fig. 20.



Required area of column $a = 2 \; (a' + a'')$ (Fig. 20 shows a complete system of latticing for this case).

The longitudinal shear between the outer and inner rib for one panel length L'' is equal to

$$S'' = \frac{SM''}{ar^2} L'' = 8\, c \frac{M''L''}{dr} \quad . \; . \; . \quad (7)$$

where M'' = Static Moment $a''e''$.

Therefore, area of outer bar required

$$A = \frac{S''}{k} \operatorname{cosec} \alpha'' = 8 \frac{c}{k} \; \frac{M''L''}{dr} \operatorname{cosec} \alpha'' \quad (8)$$

or, $\frac{1}{2}$ of this when there is latticing on two sides.

Correspondingly, we find for the latticing between the inner ribs

$$S' = 8\, c \frac{M'L'}{dr} \quad . \; . \; . \; . \; . \; . \; . \; . \quad (9)$$

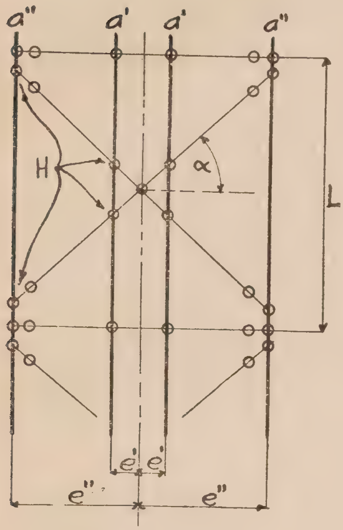
and the area of the inner bar required

$$A = \frac{S'}{k} \operatorname{cosec} \alpha' = 8 \frac{c}{k} \; \frac{M'L'}{dr} \operatorname{cosec} \alpha' \; . \quad (10)$$

where M' = Static Moment $a''e'' + a'e'$.

LATTICING OF THE LOWER CHORD (L-9) OF THE QUEBEC BRIDGE.

Fig. 21.



The top latticing as sketched in Fig. 21 (the circles indicating rivets) will first be considered.

The latticing angles between the outer and inner web practically form a complete system, only insignificant bending on the ribs being caused by the centre lines of the diagonals not meeting at the centre line of the ribs. We can apply formula (8) on page 198, to find the area which would be required in these angles, assuming hinges at points H.

Let it be assumed that the actual area has been found by the formula $16{,}000 - 70\frac{l}{r'}$, where r' is taken parallel to the webs. This area must be multiplied by $\dfrac{16{,}000 - 70\frac{l}{r'}}{16{,}000 - 70\frac{l}{r}}$ in order to find the area 'a' required for buckling laterally. The actual area is 781 square inches, $r = 19{\cdot}7$ ins., $l = 684$ ins., $r' = 16{\cdot}1$

Therefore

$$a = 781\frac{13{,}000}{13{,}600} = 746 \text{ sq. in.}$$

and $a' = a'' = \dfrac{a}{4} = 186{\cdot}5$ sq. in.

$e' = 5{\cdot}8$ in., $e'' = 27{\cdot}2$ in., $d = 67{\cdot}5$ in., $L = 73$ in.
$M'' = a''e'' = 5{,}070$.

$$S'' = 8 \times 70 \times \frac{5070 \times 73}{67{\cdot}5 \times 19{\cdot}7} = 156{,}000 \text{ lbs.}$$

Area of one diagonal required

$$A \text{ net} = \frac{1}{4}\ \frac{156{,}000}{16{,}000} \times 1{\cdot}4 = 3{\cdot}40 \text{ sq. in.}$$

$$A \text{ gross} = A \text{ net}\frac{16{,}000}{13{,}700} = 3{\cdot}97 \text{ in. gross.}$$

Actually used:

1 angle 4 x 3 x $\frac{3}{8}$ = 2·5 sq. in. gross = 1·1 sq. in. net, as one leg of one angle is cut off at the intersection at centre. Number of $\frac{7}{8}$-in. rivets required in one angle

$$N = \frac{4}{3}\ \frac{3{\cdot}40}{0{\cdot}6} = 8$$

Actual number of rivets used = 2.

Between the two inner ribs, there is no complete lattice system; the intersecting diagonals have to transmit the longitudinal shear S' of one panel length L, which can be found by formula (9).

$$M' = \frac{a}{4}(e' + e'') = 6060$$

$$S' = 8 \times 70 \frac{6060 \times 73}{67 \cdot 5 \times 19 \cdot 7} = 186{,}000 \text{ lbs.}$$

Area of one diagonal required.

$$A \text{ net} = \frac{1}{4} \frac{186{,}000}{16{,}000} \times 1 \cdot 4 = 4 \cdot 07 \text{ sq. in.}$$

Actual effective area as above = 1·1 sq. in.

Besides there are secondary strains in the lattice angles owing to their continuity, the riveted end connections and the presence of the lateral struts.

The rivets at the inner rib have to transmit the shear

$$S' - S'' = 30{,}000$$

Their number should be

$$N = \frac{1}{2} \frac{4}{3} \frac{30{,}000}{16{,}000} \frac{1}{0 \cdot 6} = 2 \text{ rivets, } \tfrac{7}{8}\text{-in. diameter.}$$

Actually used, 2 rivets $\frac{7}{8}$-in. diameter.

In consideration of these results, the lattice diagonals and their connections are decidedly too weak. It is evident that even under conservative loads certain parts must have been overstrained.

The bottom lacing is somewhat better. The tie plate at the intersection of the angles takes the longitudinal shear and is connected by 4 rivets to *each* rib.

APPENDIX D.

SECONDARY STRAINS IN TRUSSES OF QUEBEC BRIDGE.

In figuring the primary or direct strains in a truss, the truss members are assumed connected to each other by frictionless hinges. This condition is never realized; the members being either riveted and, therefore, unable to turn at their ends, or hinged, which, on account of friction, will permit only a partial turning.

When the truss deflects under the load, the angles between members tend to change. This change, however, cannot take place without bending the members at their ends, which produces bending strains in addition to the direct strains.

These bending strains are called secondary strains. On account of the labour involved in computing these strains, as they can only be determined after the trusses have been designed for the primary strains, they are considered only in rare cases; but provision for them is generally made in the adopted margin of safety.

It would be of no value to compute the secondary strains in every case, since they amount to about the same percentage of the primary strains for trusses of the same type and ordinary spans. They should, however, be carefully considered in unusual designs and in members of unusual proportions.

The secondary strains will depend largely upon the methods of manufacture and erection. In designing, the most unfavourable conditions should be considered; using, however, for the combined strains higher permissible unit strains, which may be the higher the greater the ratio of the secondary strain to the direct strain.

In order to get the maximum secondary strains for all members, different cases of loading should be considered; but generally one case, for instance, that of a total load, will suffice to show their possible magnitude. The secondary strains are the greater the deeper the member, since a bending of the ends has less effect on a slender bar than on a wider member.

As it would lead too far to give here a general theory of secondary strains, only the method followed in computing these strains in the lower chord of the Quebec bridge will be shown.

GENERAL THEORY.

The lower chord of the truss is continuous over the entire length of anchor and cantilever arms; while all the other members are pin-connected. For the present, the friction of the pins will be neglected; all the members connected to the lower chord will, therefore, be considered as turning freely at their ends and receiving no secondary strains under any load. If the lower chord sections, like the other members, were free to turn at their ends, the original angles ζ_1, ζ_2, ζ_3 (see Fig. 1) between two adjacent sections would change under a given load to $\zeta_1 + \triangle\zeta_1$, $\zeta_2 + \triangle\zeta_2$, $\zeta_3 + \triangle\zeta_3$. The change

$\Delta\zeta_3$ at panel point L_3 is equal to the sum of the changes $\Delta\alpha_1$, $\Delta\alpha_2$ $\Delta\alpha_3$, and $\Delta\alpha_4$ of the angles α_1, α_2, α_3, α_4.

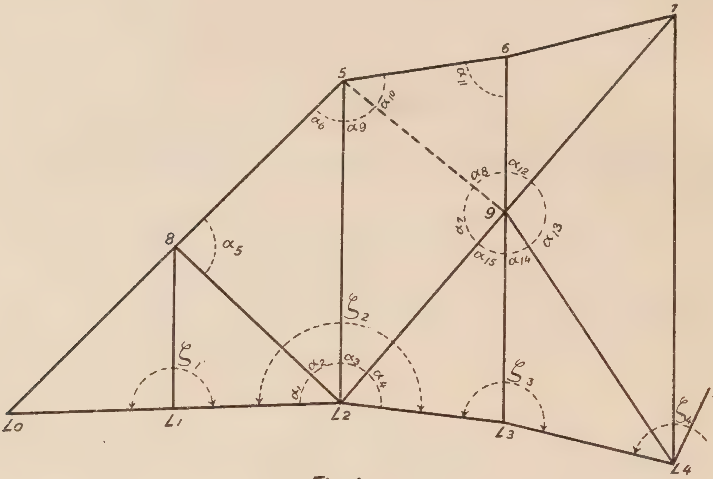


Fig. 1.

The changes $\Delta\alpha$ in any triangle of the truss, for instance, those in 2 – 5 – 8, are given by the following three equations:—

$$\left.\begin{aligned} E\,\Delta\,\alpha_2 &= (S_{8-5} - S_{8-2})\ \text{ctg.}\ \alpha_5 + (S_{8-5} - S_{5-2})\ \text{ctg.}\ \alpha_6 \\ E\,\Delta\,\alpha_5 &= (S_{5-2} - S_{8-5})\ \text{ctg.}\ \alpha_6 + (S_{5-2} - S_{8-2})\ \text{ctg.}\ \alpha_2 \\ E\,\Delta\,\alpha_6 &= (S_{8-2} - S_{5-2})\ \text{ctg.}\ \alpha_2 + (S_{8-2} - S_{8-5})\ \text{ctg.}\ \alpha_5 \end{aligned}\right\} \quad \ldots \ldots \quad (1)$$

in which S_{8-5}, S_{8-2} and S_{5-2} are the direct unit strains from the given load in the members 8-5, 8-2 and 5-2 forming the triangle, and E = Modulus of Elasticity. The change $\Delta\alpha_3$ in the trapezoid 2 – 9 – 6 – 5 is obtained as follows:—

Let the trapezoid be divided into two triangles by a diagonal 5 – 9 and apply to these triangles the above equations as follows:—

$$E\,\Delta\,\alpha_7 = (S_{2-5} - S_{2-9})\ \text{ctg.}\ \alpha_3 + (S_{2-5} - S_{5-9})\ \text{ctg.}\ \alpha_9$$
$$E\,\Delta\,\alpha_8 = (S_{5-6} - S_{6-9})\ \text{ctg.}\ \alpha_{11} + (S_{5-6} - S_{5-9})\ \text{ctg.}\ \alpha_{10}$$

from which the imaginary strain in the assumed diagonal is found:

$$S_{5-9} = \frac{(S_{2-5} - S_{2-9})\ \text{ctg.}\ \alpha_3 + (S_{5-6} - S_{6-9})\ \text{ctg.}\ \alpha_{11} + S_{2-5}\ \text{ctg.}\ \alpha_9 + S_{5-6}\ \text{ctg.}\ \alpha_{10} - E\,(\Delta\,\alpha_7 + \Delta\,\alpha_8)}{\text{ctg.}\ \alpha_9 + \text{ctg.}\ \alpha_{10}} \quad (2)$$

wherein

$$E\,(\Delta\,\alpha_7 + \Delta\,\alpha_8) = -E\,(\Delta\,\alpha_{12} + \Delta\,\alpha_{13} + \Delta\,\alpha_{14} + \Delta\,\alpha_{15})$$

This enables use to determine $\Delta\,\alpha_3$ from the triangle 2 – 5 – 9. In this way will be determined all the changes $\Delta\,\zeta$ which would take place under a given load, assuming that the lower chord sections were free to turn at their ends. Owing to the continuity of the chord, these changes in the angles ζ cannot take place without bending the chord. In other words, the forces P at the ends of each bottom chord section are no longer acting axially, but produce bending strains. (See fig. 2.)

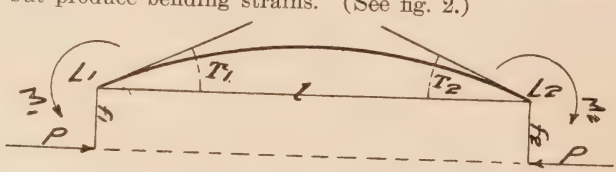


Fig. 2.

These bending strains can be obtained from the bending moments at the ends of the member $M_1 = Pf_1$ and $M_2 = Pf_2$. Between the end moments M_1 and M_2 and the angles T_1 and T_2 which the end tangents form with the original axis, l, the following relations exist:

$$\left.\begin{aligned} T_1 &= \frac{(2M_1 + M_2)l}{6IE} \\ T_2 &= \frac{(2M_2 + M_1)l}{6IE} \end{aligned}\right\} \qquad (3)$$

These formulae are obtained by integration of the differential equation of the elastic line.

$$\frac{d^2y}{dx^2} = \pm \frac{M}{IE}$$

Two adjacent lower chord members $L_1 - L_2$ and $L_2 - L_3$ (fig. 3) will now be considered. In order to have equilibrium, the two moments M_2^L and M_2^R at the panel point L_2 must be equal $= M_2$. The sum of the angles T_2^L and T_2^R must be equal to the deformation $\triangle \zeta_2$ of the angle ζ_2.

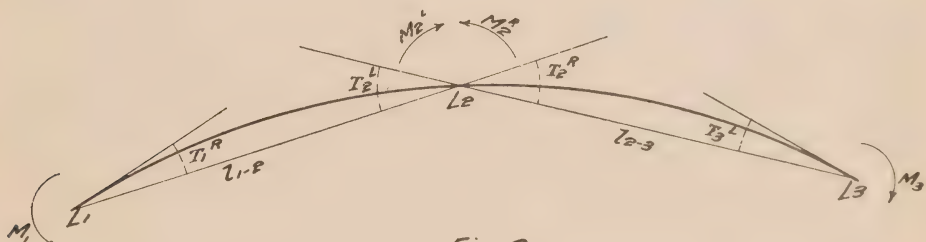


Fig. 3

$$T_2^L + T_2^R = \triangle \zeta_2 \qquad (4)$$

By substituting for T_2^L and T_2^R the values (3) it follows:—

$$\frac{(2M_2 + M_1)l_{1-2}}{6\, I_{1-2}\, E} + \frac{(2M_2 + M_3)l_{2-3}}{6\, I_{2-3}\, E} = \triangle \zeta_2$$

$$\text{or, } M_1 \frac{l_{1-2}}{I_{1-2}} + 2M_2\left(\frac{l_{1-2}}{I_{1-2}} + \frac{l_{2-3}}{I_{2-3}}\right) + M_3 \frac{l_{2-3}}{I_{2-3}} = 6\, E \triangle \zeta_2 \qquad (5)$$

Each panel point of the lower chord furnishes one equation of this kind; as many equations as there are unknown bending moments are obtained, and these moments can thus be determined. From the moments M the secondary strains in the member are found by the usual formula

$$S = \frac{Me}{I} \qquad (6)$$

wherein e = distance of extreme fibre from the neutral axis.

On account of the continuity of the lower chord, its own weight produces bending moments at the panel points, which cause bending strains in addition to the other secondary strains. If the lower chord section $L_1 - L_2$ were free to turn, it would, under

its own weight W_{1-2}, deflect like a uniformly loaded beam on two supports; the bending moment at the centre would be

$$M = \frac{W_{1-2}\ d_{1-2}}{8} \quad \ldots \ldots \quad (7)$$

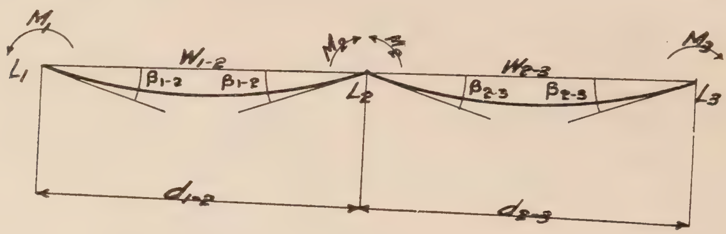


Fig. 4.

and the angles β_{1-2} which the end tangents of the elastic line form with the original axis

$$\beta_{1-2} = \frac{W_{1-2}\ d_{1-2}^2}{24\ E\ I} \quad \ldots \ldots \quad (8)$$

The angle between two adjacent lower chord sections $L_1 - L_2$ and $L_2 - L_3$ would increase by the amount

$$\triangle \zeta_2 = \beta_{1-2} + \beta_{2-3} \quad \ldots \ldots \quad (9)$$

Owing the the continuity of the chord, this increase cannot take place; therefore, bending moments will occur at each panel point. These bending moments have to correspond to equations (5) in which the values (9) have to be substituted for $\triangle \zeta$.

For the computation of the secondary strains the following cases of loading have been considered:—

1. Full dead load.
2. A load of 3,000 lbs. per lin. ft. on one truss of the cantilever arm and suspended span.
3. A load of 3,000 lbs. per lin. ft. on one truss of the anchor arm.
4. Own weight of lower chord.

The corresponding strains are given in the attached table, together with the greatest combined strains.

Under the following conditions, the secondary strains in the lower chord from dead load could practically be eliminated in the finished structure:—

1. If during erection the ends of the lower chord members were able to turn freely about the joints.
2. If after the full dead load is on the bridge, the joints would come to uniform bearing.

Both these conditions can only be partly fulfilled. Even if the lower chords were pin-connected, and the splices were not riveted up until completion of erection, friction would partly prevent turning; and it is almost an impossibility for the shop to work so accurately as to fulfil the second condition, especially for a polygonal chord like that of the Quebec bridge.

If, for instance, a butt-joint has an even bearing at the beginning of the erection, the strains would be uniformly distributed over the entire section at that time, but as soon as deformation commences, the strains will be transmitted eccentrically, causing secondary strains which may be as high as if there were no joint at all.

As it is impossible to determine the exact condition under which the joints of the lower chords come to an even bearing, it is equally impossible to ascertain what percentage of the computed secondary strains would come on any one of the chord members.

As the maximum bending moments occur at the panel points, the additional section provided for buckling may help to resist the secondary strains at the panel points where no buckling will take place.

No matter for what condition of loading the length of the members of the trusses may be adjusted to give the joints of the lower chord an even bearing, secondary strains will occur, and it is reasonable to assume that at least those produced by the live load will occur in any case. These range from 3 to 20 per cent of the total direct strains.

The total secondary strains may, therefore, range from the values S_c in the table to the values $S_c + S_d + S_w$, since S_a (from live load of anchor arm) is always of opposite sign to S_d.

The greatest secondary strain occurs in member $L_6 - L_7$ of the anchor arm, where it is between 4,600 and 22,400 lbs. per square inch.

The secondary strains in the lower chord are from 18 to 95 per cent of the corresponding direct strains; this percentage is smallest at the ends of cantilever and anchor arms, and increases towards the pier.

In figuring the secondary strains, the pins have been assumed frictionless. A calculation has shown that the strains caused in the lower chord by friction of the pins are negligible; being less than 1 per cent of the secondary strains where the latter reach the maximum.

The effect of friction of the pins is considerably greater on the eyebars of the upper chord. Approximate computations show that the secondary strains in the eyebars for assumed rigid end connections, would be from 30 to 40 per cent of the direct strains. Since for a coefficient of friction of 0·15, the strains caused by this assumed friction amount to about the same as for rigid end connections, it follows that the ends are prevented from turning under any load and the secondary strains can, therefore, amount to the above given percentages of the direct strain.

It is probable, however, that during erection as well as afterwards, through vibrations from moving loads, the eyebars gradually turn on the pins, thus eliminating partly the secondary strains from the dead load.

The most favourable condition which could be assumed is, that the secondary strains are produced by the live load only. The live load strains in the upper chord bars are from 25 to 30 per cent of the total strains; hence, if the secondary strains are 40 per cent of the primary strains produced by the live load, they will amount to at least from 10 to 12 per cent of the total direct strains.

Appendix D

Secondary Strains on Lower Chord Members of Quebec Bridge.

Notation:-

S_d = Unit Strain on extreme fibre from Dead Load.
S_c = " " " " " " a Live Load of 3000 lbs. per lin. ft. per truss on Cantilever Arm and Suspended Span.
S_a = " " " " " " " " " " " " " " " " " Anchor Arm.
S_w = " " " " " " own weight of member
+ denotes tension. The upper sign indicates the strain in the upper fibre.
− " compression. " lower " " " " " lower "
All strains given in units of 1000 lbs.

Anchor Arm.

	L_0-L_1		L_1-L_2		L_2-L_3		L_3-L_4		L_4-L_5		L_5-L_6		L_6-L_7		L_7-L_8		L_8-L_9		L_9-L_{10}.	
	at L_0	at L_1	at L_1	at L_2	at L_2	at L_3	at L_3	at L_4	at L_4	at L_5	at L_5	at L_6	at L_6	at L_7	at L_7	at L_8	at L_8	at L_9	at L_9	at L_{10}
S_d	0	± 2.4	± 2.4	∓ 3.5	∓ 2.5	± 4.7	± 4.7	∓ 9.1	∓ 7.2	± 12.2	± 12.1	∓ 13.5	∓ 13.8	± 16.6	± 16.0	∓ 14.4	∓ 14.1	± 13.6	± 12.8	∓ 7.9
S_c	0	± 2.3	± 2.3	∓ 2.6	∓ 1.8	± 2.9	± 2.9	∓ 3.6	∓ 2.8	± 4.3	± 4.3	∓ 3.1	∓ 3.0	± 4.6	± 4.4	∓ 4.0	∓ 4.0	± 4.6	± 4.3	∓ 4.1
S_a	0	∓ 1.7	∓ 1.7	± 1.9	± 1.4	∓ 1.8	∓ 1.8	± 1.2	± 1.0	∓ 0.9	∓ 0.9	∓ 0.4	∓ 0.4	∓ 0.2	∓ 0.2	± 0.2	± 0.2	∓ 0.5	∓ 0.5	± 1.1
S_w	0	± 1.1	± 1.1	± 1.1	± 0.8	± 1.2	± 1.2	± 1.3	± 1.0	± 1.2	± 1.2	± 1.2	± 1.2	± 1.2	± 1.2	± 1.2	± 1.2	± 1.2	± 1.1	± 1.4
$S_d+S_c+S_w$	0	± 5.8	± 5.8	∓ 5.0	∓ 3.5	± 8.8	± 8.8	∓ 11.4	∓ 9.0	± 17.7	± 17.6	∓ 15.4	∓ 15.0	± 22.4	± 21.6	∓ 17.2	∓ 16.9	± 19.4	± 18.2	∓ 10.6

Cantilever Arm.

	L_{10}-L_9		L_9-L_8		L_8-L_7		L_7-L_6		L_6-L_5		L_5-L_4		L_4-L_3		L_3-L_2		L_2-L_1		L_1-L_0.	
	at L_{10}	at L_9	at L_9	at L_8	at L_8	at L_7	at L_7	at L_6	at L_6	at L_5	at L_5	at L_4	at L_4	at L_3	at L_3	at L_2	at L_2	at L_1	at L_1	at L_0
S_d	∓ 7.9	± 9.4	± 10.2	∓ 10.0	∓ 10.0	± 10.6	± 11.1	∓ 9.6	∓ 9.6	± 9.0	± 9.2	∓ 5.8	∓ 6.6	± 3.9	± 3.9	∓ 2.5	∓ 2.9	± 1.9	± 1.9	0
S_c	∓ 4.1	± 3.1	± 3.4	∓ 2.6	∓ 2.6	± 3.0	± 3.2	∓ 2.5	∓ 2.5	± 2.6	± 2.6	∓ 1.5	∓ 1.7	± 1.0	± 1.0	∓ 0.6	∓ 0.7	± 0.6	± 0.6	0
S_a	± 1.1	∓ 0.3	∓ 0.3	± 0.1	± 0.1	∓ 0.0	0	0	0											
S_w	± 1.4	± 1.5	± 1.6	± 1.5	± 1.5	± 1.5	± 1.6	± 1.5	± 1.5	± 1.5	± 1.6	± 1.3	± 1.5	± 1.5	± 1.5	± 1.2	± 1.4	± 1.5	± 1.5	0
$S_d+S_c+S_w$	∓ 10.6	± 14.0	± 15.2	∓ 11.1	∓ 11.1	± 15.1	± 15.9	∓ 10.6	∓ 10.6	± 13.1	± 13.4	∓ 6.0	∓ 6.8	± 6.4	± 6.4	∓ 1.9	∓ 2.2	± 4.0	± 4.0	0